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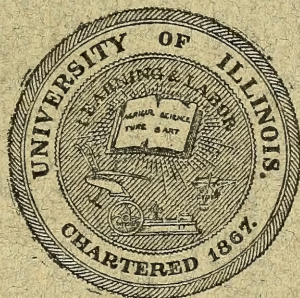
TESTS OF A LIQUID AIR PLANT

BY

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AND

C. M. GARLAND



UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

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UNIVERSITY OF ILLINOIS

ENGINEERING EXPERIMENT STATION

BULLETIN No. 21

MARCH 1908

TESTS OF A LIQUID AIR PLANT

BY C. S. HUDSON, FORMERLY INSTRUCTOR IN PHYSICS, UNIVERSITY
OF ILLINOIS, AND C. M. GARLAND, INSTRUCTOR IN
MECHANICAL ENGINEERING

The tests were made in the Mechanical Engineering Laboratory of the University of Illinois for the purpose of determining:

(a) The most economical conditions for operating the liquid air plant belonging to the departments of Physics and Mechanical Engineering.

(b) The power consumed and the cost of production of liquid air in plants of this type.

(c) The efficiencies of the separate units composing the plant.

(d) Incidentally, the keeping properties of liquid air in Dewar bulbs of different sizes, mirrored and unmirrored, enclosed in felt receptacles and open without covering of any kind.

DESCRIPTION OF APPARATUS

Fig. 1 is a diagrammatic sketch of the plant. It includes a four-stage high-pressure air compressor built by the Norwalk Iron Works, compressing up to 4000 pounds to the square inch. (See Fig. 2). The cylinders of the compressor, four in number, are arranged two in tandem, the first and third, second and fourth, and placed side by side on the bed plate. The dimensions of the compressor are: cylinders $7\frac{1}{4} \times 3\frac{3}{8} \times 2 \times 1 \times 8$ -in. stroke. The rated speed is 180 r. p. m. and the capacity 17.5 cu. ft. of free air per minute delivered. Between each compressor cylinder and the next higher cylinder an intercooler is placed, through which the air passes and in which the heat due to the compression is partly abstracted by circulating water.

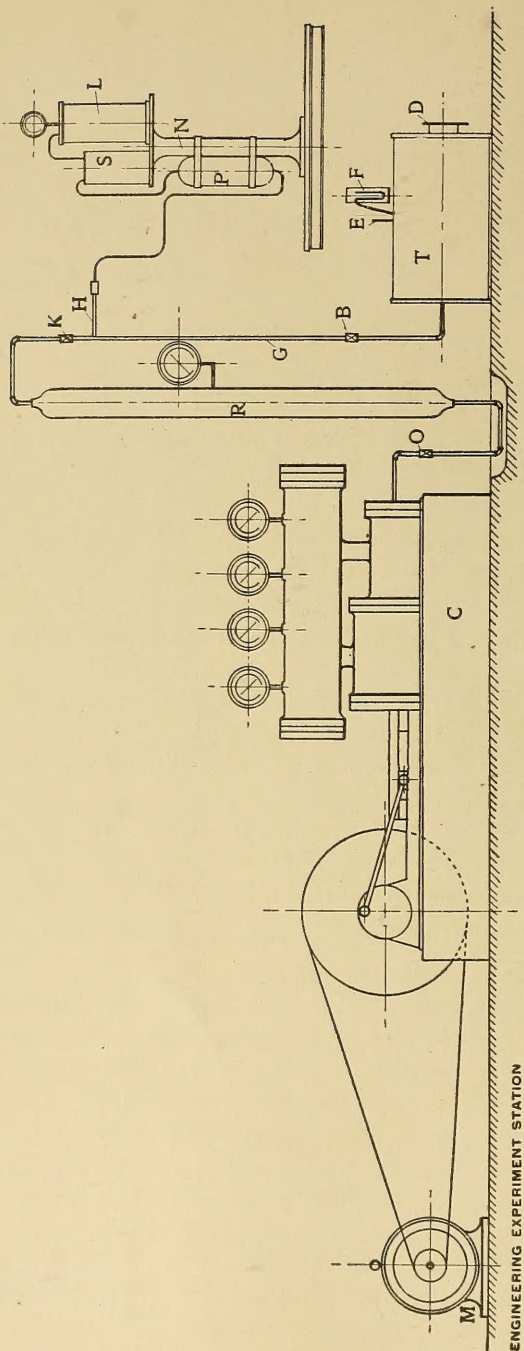
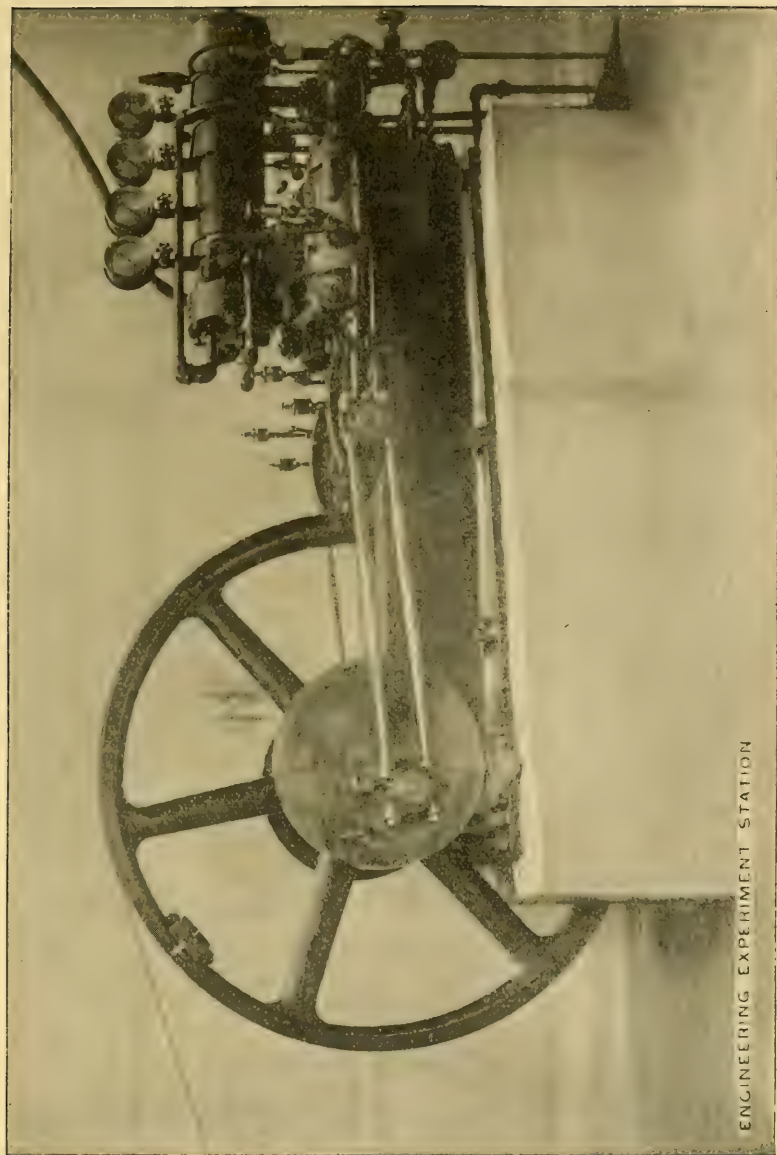


FIG. 1



ENGINEERING EXPERIMENT STATION

FIG. 2

The air for use in the compressor is ordinarily taken from the atmosphere above the laboratory building and drawn through a low pressure purifier consisting of a metallic can 15 in. in diameter by 36 in. in height, containing trays on which lime is spread. Owing to the small diameter of the air inlet, (one inch), and to a tendency of the lime to cake in the trays, this purifier has been found to be unsatisfactory, as it caused unnecessary suction on the compressor, thereby lowering the efficiency and also at times permitting lime dust to be drawn into the cylinder. During these tests this purifier was cut out and the air taken from the inside of the building. When the purifier was used again later, it was found that the greater part of the impurities, carbon dioxide and water, was removed by the high pressure purifier.

From the high pressure cylinder the air is conducted to the air receiver, *R*, (Fig. 1), through $\frac{1}{2}$ -in. extra heavy iron pipe submerged in a channel of running water. This receiver is an imported Mannesmann steel tube 12 ft. long by $9\frac{1}{8}$ in. outside diameter, tapering to about 2 in. in diameter at the end. The thickness of the walls of this tube is about $\frac{5}{16}$ in. The tube was tested by the maker to 4000 lb. per sq. in. and may be used continuously at a working pressure of 3000 lb. per sq. in. The action of the receiver in maintaining a constant pressure of the liquefier was highly satisfactory.

A 10-in. Schaefer and Budenberg hydraulic pressure gage, graduated from 0 to 8000 lb. per sq. in. in increments of 200 lb. was used to measure the pressure in the receiver. From the receiver the air is conducted through $\frac{1}{2}$ -in. iron pipe, then through $\frac{5}{8}$ -in. copper tubing to the high pressure purifier *P*, and from this through a coil of copper tubing of the same size fitted into an insulated cooling tank *S*, 8 in. in diameter by 20 in. deep. From this coil the air enters the liquefier *L*, the connections between coil and liquefier being made as short as possible in order to prevent the air from becoming warm in its passage.

The liquefier is the Hampson laboratory type, made by the Brin Oxygen Company of London. A sectional view of it is shown in Fig. 3 and 3a. The compressed air enters through the joint 36 coming directly from the coils in the pre cooler, and passing into the copper coils of the liquefier. As it circulates through these in succession, it is cooled more and more by the expanded air which rises around the coils, so that by the time it reaches

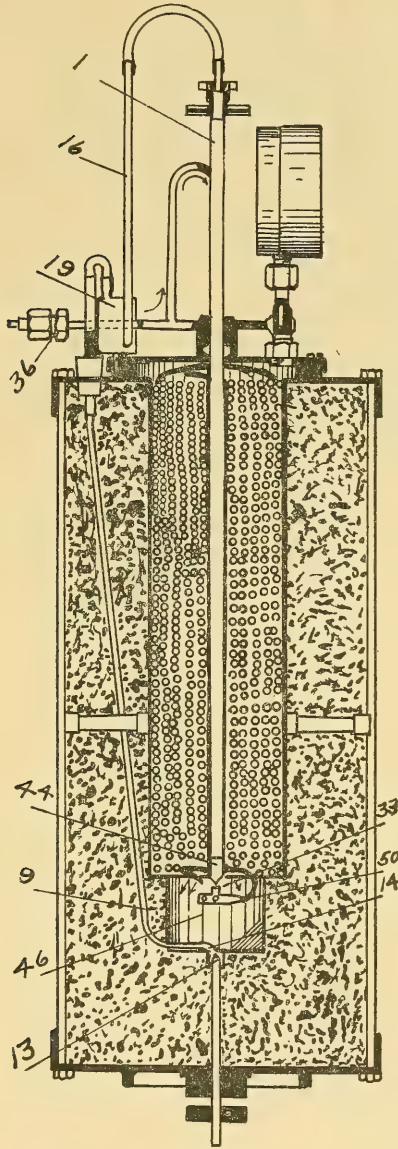


FIG. 3

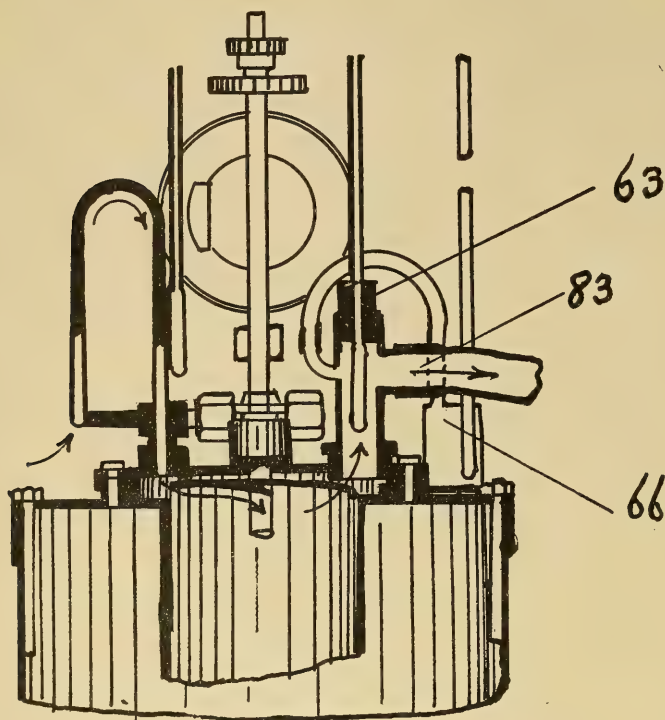


FIG. 3a

the lowest coil at the bottom, its temperature is near the liquefaction point. The air passes then through the expansion valve 33, dropping from several thousand pounds to atmospheric pressure, and in consequence some of it liquefies and collects in the brass receiver 46. The capacity of this receiver is about 130 cc. It can be emptied by turning the outlet valve at 14. The expanded air which did not liquefy rises slowly through the spaces between the coils and serves to cool the oncoming air inside the coils, as has been described above. After rising to the top of the coils this expanded air passes into tube 83 and from there proceeds back to the intake of the compressor. This expanded air does the greatest possible service if its temperature on leaving the space around the top coils at 83 is the same as the temperature of the entering compressed air, for in this case it has cooled the entering air as much as possible. In order to measure the temperature of

TABLE 1

No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
	Receiver Pressure lb. per sq. in.	Temp. Air Entering Liq. °C.	Temp. Air Leaving Liq. °C.	Room Temp. °C.	Motor Speed r. p. m.	Compressor Speed r. p. m.	Grams of Air Liquefied per hr.	Total Wt. of Air Delivered per hr. grams	Percent of Air Liquefied	Delivered horse power	Grams per d. h. p. hour	Pints Liquefied per hour	Cost of Power per Pint dollars	Total Cost per Pint dollars	Per cent d. h. p. Available in Liquid Air	
1	2000	11	9	18	938	178	1555	33700	4.6	15.40	101.0	3.53	0.288	0.39	1.5	
2	2000	19	17	16	918	173	1362	32800	4.2	15.20	89.6	3.09	0.324	0.44	1.3	
3	3088	20	18	24	1130	210	2980	39600	7.5	19.74	151.0	6.77	0.192	0.24	2.3	
4	3110	10	9	16	1139	211	3165	39800	8.0	19.50	162.0	7.18	0.178	0.23	2.4	
5	3140	4	4	19	1129	209	3170	39400	8.0	20.0	158.5	7.20	0.183	0.23	2.4	
6	3137	0	5	19	1131	210	3282	39600	8.3	20.2	164.0	7.46	0.178	0.22	2.5	
7	2128	0	4	21	1137	210	2495	39800	6.3	18.2	137.0	5.67	0.213	0.27	2.1	
8	1116	-2.5	4	21	1155	214	550	40200	1.3	13.8	39.8	1.25	0.728	1.01	0.6	

the outgoing air, a thermometer was introduced at 63. For the incoming compressed air, however, at first a steel thermometer cup was used, introduced between the purifier and the liquefier, and the temperatures were taken by means of a mercurial thermometer. This arrangement has, however, such a large conducting surface exposed to room temperature, and the heat capacity of the air flowing through it was so small, that the air was warmed to room temperature and rendered the cooling useless. After failure with this thermometer cup, a platinum copper thermoelement was soldered into the copper tubing at the entrance to the coils, the other junction being kept in ice, and with a sensitive galvanometer the temperature of the entering air was determined within the limits of about half a degree.

THE MEASUREMENT OF THE POWER

The power to drive the compressor during these tests was supplied by a 15 h. p. Westinghouse two-phase induction motor belted direct to the compressor. The electrical input to the motor was measured by means of Weston wattmeters, one in each phase of the motor circuit. After the tests were completed and before the instruments were taken from the circuit, a brake horsepower test was run on the motor. Two sets of readings were taken, one set as the load was applied, the other set as the load was taken off. Table No. 1*a* shows the results of this test. The ratio of the brake horsepower to the kilowatt input was found to be practically constant for loads from $\frac{5}{8}$ to 20 per cent overload. This ratio 0.77 was used in calculating the horsepower delivered to the compressor. This method for obtaining the delivered horsepower (d. h. p.) was used for the reason that it was impossible to calibrate the instruments in the time at our disposal.

The speed of the motor was taken by a hand counter. The compressor, as before stated, was driven by a belt from the motor. The speed of the compressor for several tests was taken by means of a mechanical counter; this, however, became unreliable, so that in the calculation of the results the speed of the compressor was calculated from the speed of the motor, there being practically no slippage of the belt.

All tests with the exception of one were of two hours' duration. Readings were taken every ten minutes. The power as given in Table No. 1, column 11, is accurate within about $3\frac{1}{2}$ per cent.

TABLE 1a

	Up	Down	Up	Down		Up	Down		
Weight on Scales pounds	Speed r. p. m.	Speed r. p. m.	Kilowatts	Kilowatts	Kilowatts Average	B. H. P.	B. H. P.	B. H. P. Average	Ratio Kw. to B. H. P.
0		1183		.8	.8				
5		1190		2.4	2.4		2.83	2.83	
10	1162	1176	3.8	3.8	3.8	5.53	5.60	5.56	
15	1153	1157	6.4	6.4	6.4	8.23	8.26	8.24	
20	1165	1140	8.4	8.4	8.4	11.09	10.85	10.97	.7650
25	1126	1135	10.4	10.4	10.4	13.40	13.51	13.45	.7738
30	1117	1126	12.2	12.2	12.2	15.95	16.08	16.01	.7624
35	1100	1100	14.4	14.8	14.6	18.33	18.33	18.33	.7963
40	1092	1082	16.0	16.2	16.1	20.80	20.60	20.70	.7780
45	1030	1030	19.2	19.2	19.2	22.16	22.06	22.06	

THE MEASUREMENT OF THE AIR USED

The measurement of the amount of air delivered by the compressor was made by means of a thin plate orifice, the necessary data being obtained from an article by Mr. R. J. Durley in the *Transactions of the American Society of Mechanical Engineers*, Vol. 27. This method is as follows: Referring to Fig. 1, *G* is a pipe line leading from the high pressure air receiver; *H* is a connection to the liquefier; *K* and *B* are special heavy globe valves; *T* is a sheet iron tank connected at one end with the half-inch pipe leading from the receiver; the opposite end of this tank is fitted at *D* with a thin

plate orifice one inch in diameter, cut from a plate of 0.0571 in. thickness. E and F are respectively thermometer and U-tube for measuring the temperature and pressure in the tank T . During the regular tests on the liquefier, the valve at B remained closed. After each test was completed, a short test was run under the same conditions of compressor speed and receiver pressure in order to determine the amount of air used, the pressure being maintained constant in the receiver by slightly opening the valve B and allowing the compressed air to expand into the tank T and escape through the orifice at D . The data recorded in these tests were the pressure in inches of water in the tank and the temperature. With these data was obtained the flow of air in the tank in pounds per second from the work of Mr. Durley as referred to above.

On looking over these tests to determine the amount of air used, it was seen that some of the results were not consistent. The discrepancy was due to the difficulty in throttling the air by means of the valve B , to the short period of the test (ten minutes), and to the fact that the pressure in the receiver varies very slowly when the escaping air has been throttled close to the proper amount necessary for the maintenance of the constant pressure. Owing to these causes and also to a desire to obtain the efficiency of the compressor, a series of tests was run on the compressor and the amount of air delivered measured in the same manner as above described. These tests were, however, from 30 to 45 minutes' duration, thus giving plenty of time for the pressure in the receiver to vary. If it was found that the pressure was slowly increasing or decreasing, the valve B was adjusted and the test was run over again. These tests, six in number, were run with pressures of approximately 1000, 2000 and 3000 lb. per sq. in. in the receiver, and with an average speed of the compressor in the first set of 200 revolutions per minute, and in the second set, of about 184 revolutions. From the results of these tests Table 2 was computed.

It will be seen from the table that the first set of tests was run at speeds of about ten per cent lower than the speeds during the regular tests on the liquefier. This was due to low voltage on the line supplying power to the motor. In order to correct for this difference in speed in calculating the amount of air supplied to the liquefier and also in calculating the per cent of air used by the liquefier, it was assumed that the air delivered was directly proportional to the speed of the compressor. This, of course, is not exactly cor-

TABLE 2

EFFICIENCY TESTS OF AIR COMPRESSOR

	Receiver Pressure	Motor Speed r. p. m.	Compressor Speed r. p. m.	Delivered horse power	Air Delivered by Compressor lb. per min.	Free Air De- livered Cu. ft. per min. at 60° F.	Mechanical Efficiency	Th'r't'c'l h. p. for Isothermal Compression	Efficiency of Compression
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TEST No. 1

1	1058	1074	198	14.4	1.380	18.13	58	5.06	61
2	2043	1084	200	18.2	1.392	18.29	66	5.90	50
3	3007	1081	200	19.9	1.386	18.21	99	6.24	45

FRICTION TEST OF AIR COMPRESSOR

1	0	1145	212	6.5=Friction h. p.					
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TEST No. 2

1	3100	1094	179	17.7	1.23	16.16	67.5	5.66	47
2	2100	1128	184	15.7	1.23	16.35	62.0	5.25	54
3	1000	1135	186	13.9	1.26	16.75	57.0	4.57	58

FRICTION TEST

1	0	1172	192	6.18=Friction h. p.					
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rect; the total error introduced, however, is quite small, and as the figures resulting from this assumption are not used in any calculations of importance, the assumption is justified.

The second set of compressor tests was run at a speed of about 184 r. p. m., obtained by changing the motor pulley. This set of tests was made in order to determine the effect of speed on the efficiency of the compressor.

It will be observed from Table 1 that the speed of the compressor during the regular tests was about 16 per cent higher than the rated speed. It was thought desirable at the beginning of these tests to run at this speed in order to operate the liquefier close to its capacity. It will be noted from Table 2 that the efficiency of the compressor falls off slightly with increasing speed, as is to be expected. This small loss in compressor efficiency is probably compensated, however, by increased efficiency of the liquefier at higher capacities.

In Table 2 will also be found the results of friction tests run on the compressor at the two speeds. The friction is practically constant and is therefore proportional to the speed. This was assumed in calculating the efficiency of the compressor, as the speeds varied somewhat owing to fluctuations in the line voltage. These friction tests were run directly after the respective compressor tests with the air inlets to the different cylinders broken so that the air was under no compression in the cylinders.

The mechanical efficiency was obtained from the formula,

$$\text{Mechanical Efficiency} = \frac{\text{Delivered h. p.} - \text{Friction h. p.}}{\text{Delivered h. p.}}$$
 The theoretical work was obtained by calculating the work of isothermal compression for the weight of *air delivered* at the pressure in the receiver. The efficiency of compression was calculated as,

$$\text{Efficiency compression} = \frac{\text{Theoretical work done}}{\text{Delivered h. p.} - \text{Friction h. p.}}$$
 and it was found to decrease with increase of pressure as was to be expected.

The amount of air delivered by the compressor as given in Table No. 2 is probably accurate within 2 per cent.

COLLECTING AND MEASURING THE AIR LIQUEFIED

At the beginning of the test, before the pressure had risen above 1000 lb., the expansion valve of the liquefier was opened

for several minutes in order to dry out the interior of the coils and to blow out whatever oil and dust had collected in them. A few drops of oil invariably came through the open outlet valve during this cleaning. The expansion valve was then closed, not to be opened again until the full pressure of the tests, 1000, 2000 or 3000 lb. was reached.

After the establishment of the desired pressure for the test, the expansion valve was again opened and the time noted which elapsed before the first drops of liquid air appeared. This interval of time is very different for the different pressures and is an important factor to be considered whenever liquid air must be quickly obtained. In the accompanying Table 3 the time necessary for the cooling from room temperature to that of liquefaction is given for the range of pressures that these tests cover.

TABLE 3

Pressure pounds	Minutes required for liquefaction			
1000	-	-	-	14.0
2000	-	-	-	6.0
3000	-	-	-	1.5

It is evident from the above table that if the pressure were reduced much below 1000 lb. the cooling would take place so slowly that no liquid air would be produced. Facts that will be given later in this article show that the lowest possible pressure at which the liquefier will yield liquid air appears to be about 900 lb.

After the liquid had begun to flow at a uniform rate, the observations of the tests were begun. The air was collected in silvered Dewar bulbs of two liters capacity, which were mounted on a platform balance directly under the outlet tube of the liquefier, which extended about an inch down the neck of the bulb. Although there is considerable evaporation from the stream of liquid air which flows from the outlet tube into the bulb, it seems impossible to prevent this loss by any simple and practical device. An attempt was made to reduce it as much as possible by making the outlet tube extend well inside the neck of the Dewar bulb. As this loss is inherent in the practical operation of this liquefier, it was thought best to charge it to the efficiency of the liquefier.

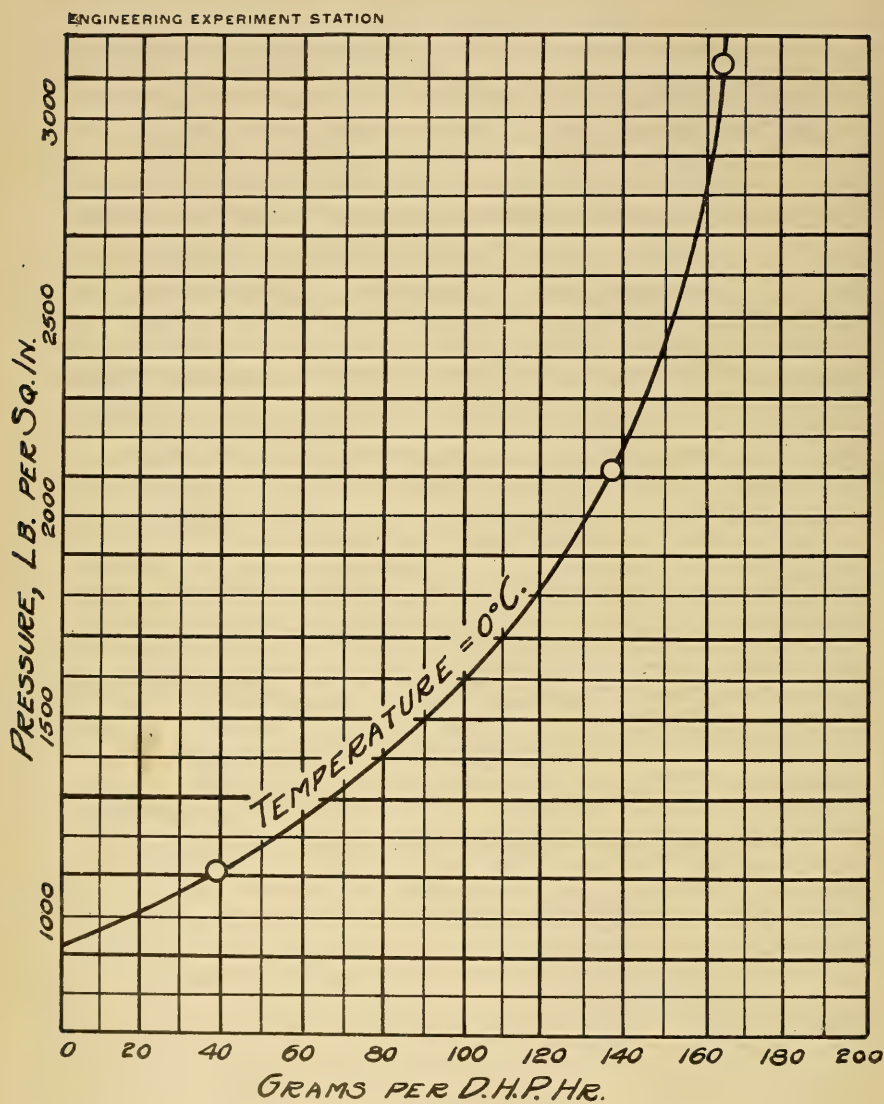


FIG. 4

The liquid which collected in the bulb was weighed at intervals of ten minutes, and the rate of production was found to be very uniform. The outlet valve was opened at intervals of about a minute during fifteen seconds in order that the liquid might run out in a full fast stream, and thus experience less loss from evaporation than if the outlet had been open continuously and the stream had been small in consequence.

The expansion valve was opened just enough so that the flow of air through it kept the pressure of the receiver down to the constant pressure of the test. It required only occasional regulation. Only twice during the eight tests did any trouble occur which was traceable to the liquefier. Once when the operator neglected to discharge the liquid air from the outlet chamber often enough it rose into the glycerine pressure gage and froze and burst the rubber tubing connection, necessitating the closing of that test twenty minutes short of the usual two hours' duration. At another time the liquefier became clogged from accumulated dirt, water and grease and had to be taken apart and all the valves thoroughly cleaned with a jet of steam. It should be stated, however, that this was the first cleaning that the liquefier had needed during its intermittent but rather hard service of eighteen months.

INFLUENCE OF PRESSURE ON THE EFFICIENCY OF THE LIQUEFIER

In Fig. 4 is shown the relation between the working pressure and the quantity of air liquefied per horsepower hour, taken from tests No. 6, 7 and 8 of Table 1, all of which were conducted at or near 0° C. It is clear from the curve that the efficiency is greater the higher the pressure. It is also evident that the efficiency would become zero, i. e., that no liquefaction would take place, at a pressure of approximately 900 lb. per sq. in. Also, increase of pressure at high pressures causes a smaller increase in efficiency than does the same increase in pressure at low pressures. This last conclusion shows that it is not desirable to increase the working pressure much over 2500 lb. per sq. in. in producing liquid air with this liquefier, for the reason that only a small gain in efficiency is thereby obtained, and this at a cost of a very considerable increase in the trouble and danger attendant upon the maintaining of high pressures in the plant. The numerous con-

nections and valves of the plant must be continually inspected in order to prevent serious leaks when the pressure is 3000 lb., but at lower pressures there is little difficulty in this respect.

Our results regarding the influence of pressure on the efficiency of the liquefier agree with the conclusions of other experimenters of this general type of liquefier. Others¹ have, however, found that the lowest pressure at which liquefaction occurs is about 700 lb.

INFLUENCE OF TEMPERATURE ON THE EFFICIENCY OF THE LIQUEFIER

The temperature of the air as it enters the coils of the liquefier has an important influence on the efficiency of the liquefier, the lower the temperature the greater being the efficiency. We have measured the influence of temperature over a range between 0° and 20° C. for two pressures, namely 2000 and 3000 lb. per. sq. in., and the data are given in Table 1 under tests 1 to 7 inclusive. In the accompanying Fig. 5 these results are plotted. The curves for the two pressures closely approximate straight lines. If the inlet temperature were below 0 the efficiency would be correspondingly greater and the cost of production less; but it does not seem advisable in intermittent operation of the plant to reduce the temperature of the entering air below 0, as the trouble and labor necessary to keep a freezing mixture of salt and ice in the precooler more than balance the gain in efficiency. Indeed, if cold tap water of a temperature of about 15° is allowed to run through the precooling tank, the efficiency at 2500 lb. is almost as great as would be obtained ordinarily when cracked ice is placed in the tank. The reason for this apparent contradiction is that unless the ice is continuously stirred, the water next to the cooling coils becomes warmed much above 0° C., but if running water flows through the tank its motion keeps cold water near the coils. The temperature of the coil is, therefore, not much different whether unstirred ice or running tap water is used; the latter method of cooling, however, is more convenient and less expensive. In our tests at 0° C. a mixture of ice and salt was used, being stirred every five minutes; the temperature of the air was not reduced much below 0° C. and in all cases it was directly measured by the thermojunction at the entrance to the liquefier.

¹ Bradley and Rowe, *Physical Review*, Vol. 19, 1904, p. 330.

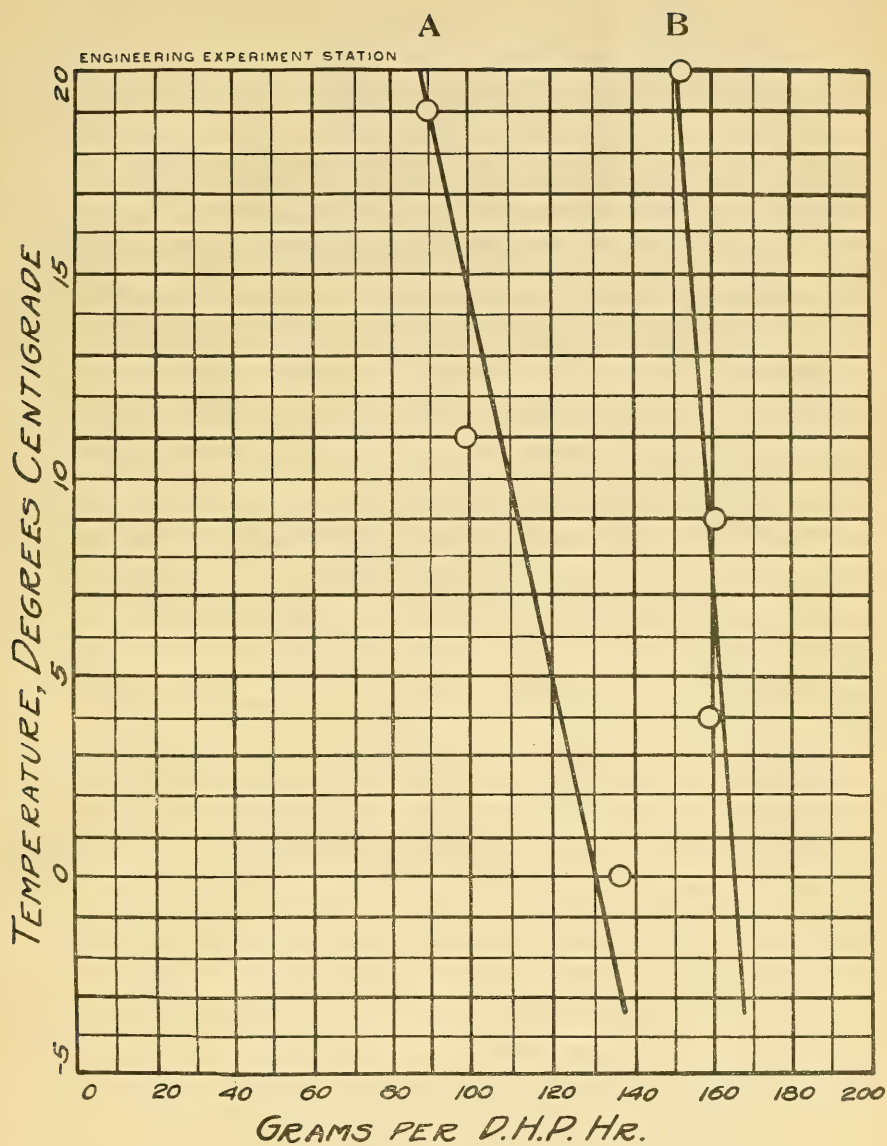


FIG. 5

A = 2000 lb. per sq. in.
B = 3000 lb. per sq. in.

It will be seen from Fig. 5, that when the liquefier is working at high pressure, the influence of temperature on efficiency is not nearly so marked as at lower pressures. This effect is quite marked, and leads to the conclusion that if a high pressure is used, there is little need to pay much attention to precooling. As high pressure is of itself an aid to efficiency, this extra beneficial influence which accompanies high pressures is doubly important. Its explanation seems to be somewhat as follows: The cooling effect at the expansion valve per unit weight of air passing through is approximately proportional to the drop in pressure at this valve. It is, therefore, greater for 3000 lb. than for 2000 lb. working pressure. But the heat which must be abstracted from the air which is eventually liquefied is reduced when the temperature of the entering air is reduced, and therefore the cooling effect at the expansion valve produces more liquefaction when the inlet temperature is low. Further, when the pressure is low, the cooling at the expansion valve is less than it is when the pressure is high, and, therefore, the change in temperature of the entering air affects the yield at low pressures to a greater extent than it does at the higher pressures. This explains why the two curves of Fig. 5 have different slopes.

THE LOSS OF ENERGY IN PRODUCING LIQUID AIR

Liquid air is a source of considerable available energy on account of the expansive force that it exerts when it is warmed to room temperature. This available energy is obviously equal to the heat required to vaporize the liquid air at its boiling point plus that required to raise its temperature to that of the room; an amount which is known to be in total 97.5 gram calories per gram liquid air.¹ In column 16 of Table 1 are given the ratios of the available energy of the liquid air to the energy received by the compressor in the production of the liquid. It is seen that under the most favorable conditions the available energy of the liquid air is only 2.5 per cent of that expended in producing liquefaction. This means that if the liquid air were used as the motive power in an engine, the work which it could perform under the most favorable conditions is only $2\frac{1}{2}$ per cent of the work which was required to produce the liquid air. It is thus very clear that liquid air is wholly unsuited to the storage or transfer of power.

1 Allen and Ambler, *Physical Review*, Vol. 15, p. 183, 1902.

THE KEEPING OF LIQUID AIR IN DEWAR BULBS

A few tests were made in order to learn the approximate rate of evaporation of liquid air from Dewar vacuum bulbs under varying conditions of size of bulb, presence of felt coverings, and silvering of the bulb. Previous experimenters have found that similar bulbs show very different behavior, probably caused by the varying of the thickness of the silvering, which is a good metallic conductor, and the different degrees of exhaustion and heat conductivity of the glass bulbs.

Regarding the influence of the size of the bulb on the rate of evaporation, no important difference was noted between the proportionate loss from two silvered bulbs, one of a liter capacity and half filled, the other of a quarter liter capacity and filled; both flasks lost one quarter of their liquid air in 24 hours when exposed to the temperature of 20° C. unprotected by any coverings. Covering the bulb with felt diminished the loss by evaporation, for a silvered liter flask containing at the start 900 grams of liquid air was found to preserve 100 grams after standing four days in its felt-lined basket, which is a rate of loss somewhat less than that for the uncovered flasks.

But a most noticeable aid in the protection of the air from evaporation comes from the silvering of the bulbs, which reflects light and radiant heat that would otherwise be absorbed by the bulb and its contents. The loss from unsilvered but evacuated bulbs is so great that a boiling of the liquid air in them can be seen, whereas the silvered bulbs show a quiet surface when viewed through the mouth of the bulb. In an experiment on two similar liter bulbs, one of which was silvered, the other not, it was found that after standing 24 hours all of the half liter of air in the unsilvered bulb had evaporated, but only 100 grams of the half liter in the silvered bulb.

THE COST OF PRODUCING LIQUID AIR

In columns 14 and 15 of Table 1, is given the cost of production of liquid air as given by these tests. The only expenses which are here included are the cost of the power used, estimated at eight cents per kilowatt hour, and the cost of an attendant, reckoned at thirty-five cents per hour.

When the plant is running at 3000 lb. pressure and the temperature of the air entering the liquefier is 0° C., conditions which give the greatest efficiency that we have obtained, the total cost of the liquid air, including both the cost of the power and of the operator's services, is 22 cents per pint. When the plant is running at 15° C. and 2500 lb. pressure, which are the most convenient conditions of temperature and pressure, the total cost is about 32 cents.

PUBLICATIONS OF THE ENGINEERING EXPERIMENT STATION

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BULLETIN NO. 22

TESTS OF CAST-IRON AND
REINFORCED CONCRETE
CULVERT PIPE

BY

ARTHUR N. TALBOT



UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

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UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

BULLETIN No. 22

APRIL 1908

TESTS OF CAST-IRON AND REINFORCED CONCRETE
CULVERT PIPE.

BY ARTHUR N. TALBOT, PROFESSOR OF MUNICIPAL AND SANITARY
ENGINEERING AND IN CHARGE OF THEORETICAL
AND APPLIED MECHANICS.

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I. INTRODUCTION.

1. *Scope of the Bulletin.*—The use in recent months of concrete and reinforced concrete pipe for culverts in railway embankments has brought to the minds of engineers anew the question of the action of a pipe when subjected to the external pressure or load of an embankment and of its resistance to this external pressure. The subject is somewhat related to the action of sewers under earth pressure in a trench. The question of distribution of loads and pressures in a trench or embankment is varied and complicated, depending as it does upon the variety and conditions of the earth and the manner of the filling. The laws of the pressure of earth of themselves would require an extensive investigation and treatment, and this phase is not taken up here. The main tests described were made with a specially prepared testing apparatus which included a box of strong and stiff construction, and the pipes were embedded in sand and the load applied through a saddle which rested on a sand cushion. The results throw light upon the resistance of pipe to embankment pressures and also upon the action of sewers under similar conditions. Cast-iron pipes, concrete pipes, and reinforced concrete pipes were tested. Auxiliary tests were made to connect the results of the investigation with the strength of the materials. Thus, rings which were cut from the spigot end of the cast-iron pipe were tested under concentrated load, and small test specimens cut from the pipe were tested in cross bending. Short rings of concrete and reinforced concrete were tested, both under concentrated load and under distributed load. In different ways these auxiliary tests served to check up the phenomena of the tests of the pipes and to assist in interpreting the action of the testing apparatus, the distribution of the load, and the resisting strength of the structures themselves. In planning the tests the relation of the various phenomena was kept in mind, and the results are compared and discussed.

2. *Acknowledgment.*—The investigation was made possible through the co-operation of five railroad companies: Atchison, Topeka & Santa Fe Ry. System; Chicago, Burlington & Quincy R. R. Co.; Chicago, Milwaukee & St. Paul Ry. Co.; Chicago, Rock Island & Pacific Ry. Co., and Illinois Central R. R. Co. These companies furnished the cast-iron pipe and the reinforced concrete pipe for the tests. Acknowledgment is due to C. H. Cartlidge, Bridge Engineer C., B. & Q. R. R.; A. F. Robinson, Bridge Engineer A., T. & S. F. Ry.; C. F. Loweth, Engineer and Superintendent, Bridges and Buildings, C., M. & St. P. Ry.; J. B.

Berry, Chief Engineer C., R. I. & P. Ry., and R. E. Gaut, Bridge Engineer I. C. R. R., for the assistance they gave in arranging for furnishing the pipes. The tests were made between November, 1906, and January, 1908. The work was an outgrowth of the thesis investigation of W. A. Slater ('06, municipal and sanitary engineering) upon the strength of concrete rings. Part of the experimental results of Mr. Slater are included, and a reference is made to the thorough analytical treatment made by Mr. Slater along the lines of Bach's method for curved beams. The 1907 tests of concrete and reinforced concrete rings were part of the thesis work of Messrs. H. B. Bushnell, J. Cermak and A. P. Poorman, senior students in civil engineering, Class of '07. The rings used in these tests were made by employees of the Engineering Experiment Station. Supervision of the tests and assistance were also given. The main tests were made by the Engineering Experiment Station. Special acknowledgment is made to D. A. Abrams, Associate, and W. R. Robinson, First Assistant in the Engineering Experiment Station for assistance in these tests and in the preparation of this bulletin.

II. MECHANICS OF PIPES AND RINGS SUBJECT TO EXTERNAL PRESSURE.

3. *Bending Moment and Conditions of Loading.*—The stresses developed in rings subject to external earth pressure, as in sewers and railroad culvert pipes, are of course dependent upon the bending moments developed, and, as the exact load coming upon the ring and its distribution over the surface are difficult to determine, the bending moment is in general quite uncertain. The amount of the load and its distribution, and therefore the bending moments on different parts of the ring, depend upon a number of conditions, among them the nature of the earth used in the filling, the method of bedding the pipe, the way of tamping the earth at the sides, the amount of the lateral restraint or pressure of the earth horizontally, the method of filling and packing the earth above, the condition of moisture in the earth, etc. Evidently in such earth as saturated quick-sand, the conditions may approach those of external hydrostatic pressure, and on the other hand, in deep sewer trenches, the earth filling may act in such a way that much of its weight is carried against the sides of the trench. In discussing the stresses in rings, it may be well first to find the bending moment for certain assumed conditions of loading, then to make tests under various conditions of loading, and finally to compare these results with a view of determining the probable range of bending moments under the actual conditions

of construction. The assumed loadings may include (1) a concentrated load at the crown of the ring, (2) a vertical load distributed uniformly over the horizontal section, (3) a distributed vertical load together with a horizontal load distributed vertically over the sides of the ring, and (4) an oblique loading. In these calculations, since much uncertainty is involved, the difference in the intensity of the load at the crown and at the extremities of the horizontal diameter, due to the different depths of earth, need not be considered. In general the pressures and distribution on the lower half of the ring will be considered to be the same as on the upper half. It is apparent that in a ring of considerable thickness in comparison with its diameter there is a different distribution of stresses from that found in thin rings, but for the rings under consideration the simplicity of analysis for thin rings will outweigh the small loss in accuracy. The possible modifications and complications in the analysis of thick rings may also be considered. As refinements are not essential and approximations are permissible, the analysis will assume a thin ring of homogeneous material having a constant modulus of elasticity and it will also be assumed that the changes from a circular form will have little effect upon the dimensions of the ring.

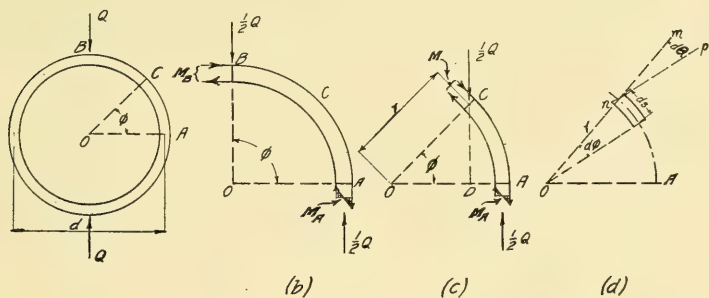


FIG. 1. RING UNDER CONCENTRATED LOAD.

4. *Concentrated Vertical Load on Thin Elastic Ring.*—Consider that a concentrated load Q is applied along the top element of a cylindrical ring and that the ring is supported along an element at the bottom, as indicated in Fig. 1(a). Since the ring is a continuous curved beam, the analysis will require a slight modification of the convention commonly used for simple straight beams. However, in any segment of the ring, the external forces acting on the ring will be held in equilibrium by the internal or resisting forces acting upon this segment at its

two ends. The moment of the internal forces acting at right angles to a section of the ring at an end of the segment is the resisting moment developed, and the bending moment may be considered to be an equal moment having the opposite sign. If we take a quadrant of the ring, as shown in Fig. 1 (b), it is evident from a consideration of the external and internal forces acting upon this ring that this quadrant will be in equilibrium under the action of $\frac{1}{2}Q$ at B, a reaction or thrust of $\frac{1}{2}Q$ at A, a resisting moment in the section of the ring at A which we will call M_A , and a resisting moment in the section of the ring at B which we will call M_B . The amounts of the two resisting moments so developed and thus of the two bending moments it is important to determine. Similarly, if we consider a portion of the ring shown in Fig. 1(c), the forces which hold it in equilibrium may be shown to be $\frac{1}{2}Q$ at A, $\frac{1}{2}Q$ at C, the moment M_A at the section A, and a variable moment M at C, the value of which will change with a change in the angle ϕ . Taking moments about A, the following equation for the value of the bending moment at any point on the ring results:

$$M = \frac{1}{2}Qr (1 - \cos \phi) - M_A \dots\dots\dots (1)$$

When M_A is known, the value of M may be determined for any point in the ring. The bending moment at A, as is shown by making $\phi = 0$ in equation (1), is $-M_A$.

When a straight beam deflects under load its axis becomes a curve of varying curvature. Each normal section of the beam will change direction through an angle which we will call θ , and at each point there will be a definite radius of curvature. By the common theory of flexure the following equation is true for straight beams:

$$\frac{M}{EI} = \frac{1}{R}$$

where R is the radius of curvature of the elastic curve, E is the modulus of elasticity of the material, I is the moment of inertia of the cross section of the beam, and M is the bending moment at the point considered. By calculus we know that the reciprocal of the radius of curvature is

equal to $\frac{d\theta}{ds}$, this derivative here expressing the rate of change of the

direction of the normal to the elastic curve from its original position with respect to the length of curve in which the change is made. Substituting this,

$$\frac{M}{EI} = \frac{d\theta}{ds} \dots\dots\dots (2)$$

This equation is ordinarily applied to the straight beam, but it is general and is therefore applicable to the elastic ring, if we consider that $d\theta$ refers to the angular change of the normal relatively to its original position. Substituting for ds its equal $rd\phi$ (Fig. 1(d)), where r is the mean radius of the ring, the equation takes the form

$$\frac{M}{EI} = \frac{d\theta}{rd\phi} \dots\dots\dots (3)$$

This may be conveniently used in form $d\theta = \frac{Mr d\phi}{EI}$. It may help

to a comprehension of the significance of the expression $\frac{d\theta}{d\phi}$ to study the representation of the elements in Fig. 1(d). The line OM shows the change in position of a section at the end of this element due to the thrust or pure compression and the line NP that due to the bending moment. The angle $d\theta$ is the resulting change in the direction of the normal due to the bending moment in the length ds , and the ratio at

the limit becomes $\frac{d\theta}{rd\phi}$. $\frac{d\theta}{d\phi}$ then represents the rate of change of the

deflected normal with respect to the angle ϕ passed over. For our purposes we may consider that r remains constant in equation (3).

It is plain that for the part of the ring between A and B in Fig. 1(b), whatever may be the local changes in directions at the various points along the quadrant, the tangent at A remains constantly vertical and the tangent at B remains constantly horizontal, and therefore the total change in θ between A and B is zero and hence that

$$\int_0^{\frac{1}{2}\pi} \frac{d\theta}{d\phi} = 0 = \int_{M_A}^{M_B} \frac{Mr}{EI}.$$

The general value of M , applicable to any point on the quadrant, must of course be used in this expression. Substituting the value of M from equation (1), using the modified form of equation (3) and integrating with respect to θ , we have

$$\int_0^{\frac{1}{2}\pi} \frac{Qr}{2} (1 - \cos \phi) d\phi - \int_0^{\frac{1}{2}\pi} M_A d\phi = \frac{Qr}{2} (\frac{1}{2}\pi - 1) - M_A \frac{1}{2}\pi = 0$$

$$\therefore M_A = \frac{Qr}{2} (1 - \frac{1}{\frac{1}{2}\pi}) = .091 Qd \dots \dots \dots (4)$$

where d is the mean diameter of the ring.

Substituting this value of M_A in equation (1) and making $\phi = 90^\circ$, we have

$$M_B = \frac{Qd}{4} - .091 Qd = .159 Qd \dots \dots \dots (5)$$

It will be seen that the bending moment at B is about sixteen-ninths times that at A.

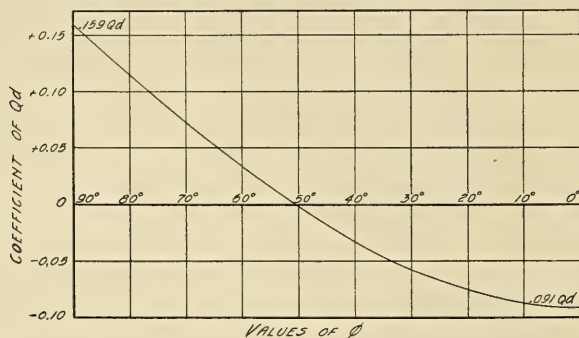


FIG. 2. VARIATION IN BENDING MOMENT FOR CONCENTRATED LOAD.

To determine the point of zero bending moment place equation (1) equal to zero. $\cos \phi = .636$ and $\phi = 50^\circ 30'$. At this point the algebraic sign of the bending moment changes from negative to positive.

Fig. 2 gives the variation of the bending moment from A to B.

It may be shown that if the load be applied equally at two points on either side of the crown (and similarly supported below) the bending moment at the crown will be decreased and that if these points are immediately above the quarter points of the diameter the value of the bending moment at the crown becomes $0.054 Qd$ and that at the extremities of the horizontal diameter $0.071 Qd$. This has a bearing upon the effect of the methods of bedding a pipe.

5. *Distributed Vertical Load on Thin Elastic Ring.*—Consider that the vertical load is distributed uniformly over the horizontal projection of the ring, as shown in Fig. 3(a), and call w the load per lineal unit of horizontal width for a ring one unit long and r the mean radius of the ring. The conditions for a quadrant segment AB are indicated in Fig. 3(b). It is easily shown that this segment is in equilibrium under the resultant of the load wr applied at D, an equal thrust or reaction wr applied at A, a moment in the section of the ring at A which

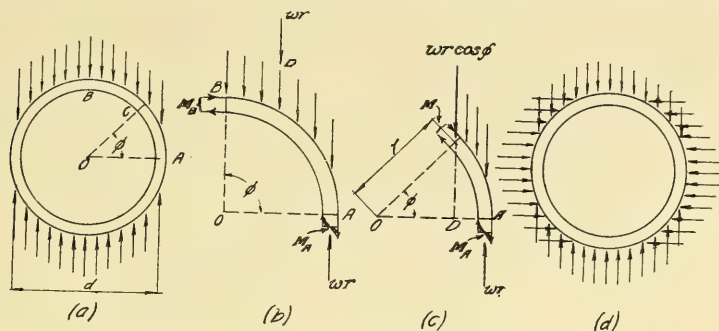


FIG. 3. RING UNDER DISTRIBUTED VERTICAL LOAD.

we will call M_A and a moment in the section of the ring at B which we will call M_B . It is seen that there is no thrust and no shear at B. Similarly the segment AC is in equilibrium under the forces and moments shown in Fig 3(c). The vertical force at C is the resultant of the tangential thrust and normal shear at that point, and is equal to the load applied between C and the crown of the ring. The expression for the bending moment M at any point C on the ring is found by taking moments about C.

$$\begin{aligned} M &= wr^2 (1 - \cos \phi) - \frac{1}{2} wr^2 (1 - \cos \phi)^2 - M_A \\ &= \frac{1}{2} wr^2 (1 - \cos^2 \phi) - M_A \dots \dots \dots (6) \end{aligned}$$

The general relation $\frac{Mr}{EI} = \frac{d\theta}{d\phi}$ (equation (3), p. 7) used in the discussions of moments for concentrated vertical load is applicable to this loading. Here again $\frac{d\theta}{d\phi}$ measures the rate of angular change of the normal to the elastic curve, and since the total angular change in curvature between A and B is zero, the tangents at A and B do not

change from their original position. We may then substitute the value of M from equation (6) in equation (3). Putting it in the modified form, equating it to zero and integrating with respect to θ , with the assumption that E , I , and r are constants, we have the following:

$$\int_0^{\frac{1}{2}\pi} wr^2 (1-\cos \phi) d\phi - \frac{1}{2} \int_0^{\frac{1}{2}\pi} wr^2 (1-\cos \phi)^2 d\phi - M_A \int_0^{\frac{1}{2}\pi} d\phi = 0$$
$$\therefore -M_A = M_B = \frac{1}{16} wd^2 = \frac{1}{16} Wd \dots\dots\dots (7)$$

where d is the mean diameter of the ring and W is the total load on a ring of unit length.

A very neat determination of the value of M_A and M_B has been given as follows: If a system of horizontal forces equal to the vertical forces here considered be applied to a ring, the bending moment produced at A by the horizontal forces will be the same as that produced at B with the vertical load, and the bending moment produced at B will be the same as that found at A with a vertical load, but with opposite signs in each case. Similarly, at any point between A and B it seems evident that an equal numerical bending moment will be produced with the new loading as at corresponding points with the old loading, but with opposite signs. The effect of a combination of this vertical and horizontal loading (Fig. 3 (d)) will be the same as that of a load normal to every part of the ring, thus producing pure compression in every part of the ring and making the bending moment at every section zero. It follows then that

$$M_A = M_B = \frac{1}{16} Wd.$$

To find the point of zero bending moment make equation (6) zero. This gives $\phi = 45^\circ$. Above this point the bending moment is positive and below it negative.

Fig. 4 shows the variation of the bending moments between A and B.

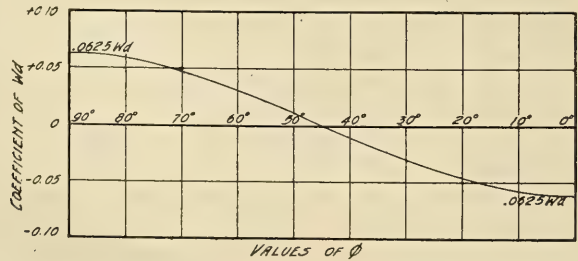


FIG. 4. VARIATION IN BENDING MOMENT FOR DISTRIBUTED LOAD.

6. *Distributed Vertical and Horizontal Loads on Thin Elastic Ring.*—Let us consider that the vertical load is distributed over the horizontal section of the pipe as before (w per lineal unit of width of pipe) and that there is a horizontal pressure uniformly distributed vertically against the pipe, the amount of this horizontal pressure per lineal unit of vertical distance being qw , where q is the ratio of the horizontal to the vertical intensity of pressure. The conditions are indicated in Fig. 5 (a). We may consider that the effect of these loads is the combined effect of the two loads. Call M' , M'_A , and M'_B the bending moments produced by the vertical load and M'' , M''_A and M''_B , the bending moments produced by the horizontal load. The bending moment at any section C (Fig. 5 (b)) produced by the vertical load is

$$M' = wr^2 (1 - \cos \phi) - \frac{1}{2} wr^2 (1 - \cos \phi)^2 - M'_A.$$

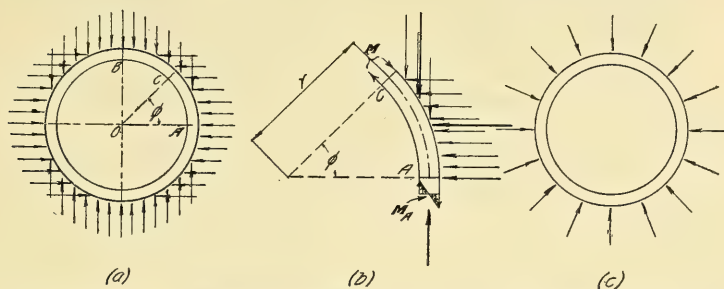


FIG. 5. RING UNDER DISTRIBUTED VERTICAL AND HORIZONTAL LOAD.

It may be shown that the bending moment produced at any section C by the horizontal load is

$$M'' = -\frac{1}{2} qwr^2 \sin^2 \phi + M''_A$$

and that the value of M''_A is $\frac{1}{4} qwr^2$. The resulting moment therefore is

$$M = M' + M'' = \frac{1}{4} wr^2 [1 + q - 2 \cos^2 \phi - 2q \sin^2 \phi] \dots (8)$$

The moment at B and A will therefore be

$$M_B = -M_A = \frac{1}{4} (1 - q) wr^2 = \frac{1}{16} (1 - q) Wd \dots (9)$$

where W is the total vertical load on the ring. The bending moment becomes zero at $\phi = 45^\circ$ as in the other case.

If the intensity of the horizontal pressure is the same as that of the vertical pressure, $q = 1$ and M becomes zero at all points. This corresponds to uniform external pressure as shown in Fig. 5(c) and produces equal compression in all parts of the ring.

7. *Oblique Load on Thin Elastic Ring.*—In the case of both the concentrated load and the distributed load it is seen that the bending moments at A and B are large and that the moment decreases to zero amount at some point between A and B. If we were sure that this loading obtained, no provision against bending need be made at the points of zero moment and but little for points close on either side. However, it must be borne in mind that if there is a change from the specified loading, the conditions of the bending moment are likewise changed. If, for example, the method of filling over the pipe should be such as to make the pressure come obliquely as shown in Fig. 6 (a),

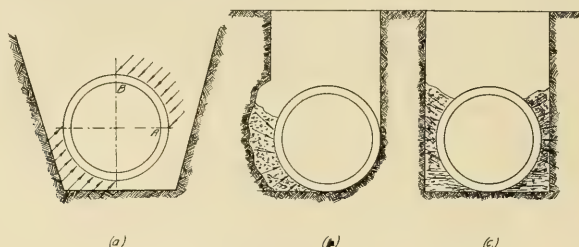


FIG. 6. VARIETIES OF LOADING.

the maximum bending moment would be at the 45° points and the minimum moments at the ends of horizontal and vertical diameters. Similarly, if in a sewer trench, a slip of earth from the side caused the pressure to come against the sewer as shown in Fig. 6 (b), the distribution and amount of the bending moment would be materially different from that of the vertical loading usually assumed. In the case of a large sewer in a shallow trench, during the time of filling, especially with the concrete still green, the direction of pressures indicated in Fig. 6 (c) would give bending moments quite unlike those before described. It will be necessary to provide separate analyses for such cases. While an accurate measure of the bending moments in such cases is impossible, yet in any case it is feasible to judge of the amount and location of the bending moments within reasonable limits and to provide strength in the section of the sewer to take the consequent stresses.

8. *Resisting Moment and Calculation of Stresses.*—For a ring whose thickness is small in comparison with the diameter the difference in the length of the inner fiber and outer fiber is small and the expression for the resisting moment given for ordinary straight beams may be applied with a close degree of approximation. In the following form-

ulas the length of the ring (width of beam) will be considered unity. Call t the thickness of the ring.

For the rectangular section of the ring the resisting moment will then be $\frac{1}{6} ft^2$ where f is the unit-stress at the remotest fiber. In those sections where there is no thrust, the maximum stress (stress at the remotest fiber) may be found by equating the expression for the resisting moment and the expression for bending moment given by equations (1), (5), (6), (7), etc., and substituting the numerical values at the section considered. If a thrust exists at the given section, this thrust may be considered to be uniformly distributed over the section and the stress will be equal to the sum or difference of the resisting moment stress and the thrust stress.

For a concentrated load at the crown (Fig. 1) the stress at B, since there is here no thrust, may be determined from the formula

$$\frac{1}{6} ft^2 = M_B = 0.159 Qd \dots \dots \dots (10)$$

where Q is the concentrated load applied at the crown and d is the mean diameter of the ring. The maximum tensile stress and the maximum compressive stress at this section will be equal. As this is the section of greatest bending moment and as the tensile strength usually governs the strength of the rings under consideration, this equation is the one to be used in tests with a concentrated load.

At A the same form of expression may be used for the resisting moment, but this must be combined with the stress due to the vertical thrust, $\frac{1}{2} Q$. Considering this thrust to be uniformly distributed, the stress in the remotest fibers will be

$$f = \frac{1}{2} \frac{Q}{t} \pm \frac{M_A}{\frac{1}{6} t^2} = \frac{Q}{t} \mp \frac{0.091 Qd}{\frac{1}{6} t^2} \dots \dots \dots (11)$$

The $-$ sign will be used for the outer fiber and the $+$ sign for the inner fiber.

At any point C (Fig 1 (c)) the stress at the remotest fiber may be shown to be

$$f = \frac{1}{2} \frac{Q \cos \phi}{t} \pm \frac{M}{\frac{1}{6} t^2} \dots \dots \dots (12)$$

For a uniformly distributed horizontal load the stress at the crown B will be, calling W the total distributed load on a ring of unit length and d the mean diameter of the ring,

$$f = \frac{M_B}{\frac{1}{6} t^2} = 1/16 \frac{Wd}{\frac{1}{6} t^2} \dots \dots \dots (13)$$

and at A

$$f = \frac{1}{2} \frac{W}{t} \pm \frac{M_A}{\frac{1}{6} t^2} = \frac{1}{2} \frac{W}{t} - \frac{3}{8} \frac{Wd}{t^2} \dots\dots\dots (14)$$

and at any point C

$$f = \frac{wr \cos^2 \phi}{t} \pm \frac{M}{\frac{1}{6} t^2} \dots\dots\dots (15)$$

For rings in which tensile stresses control, the weakest section is at the crown, and equation (13) may be used.

For a distributed vertical and horizontal load (Fig. 5) there will be a thrust at both A and B. The stresses at the crown B will then be given by the following equation, calling q the ratio of the horizontal intensity of the load to the vertical intensity,

$$f = \frac{qwr}{t} \pm \frac{\frac{1}{16} Wd}{\frac{1}{6} t^2} \dots\dots\dots (16)$$

and at A the extremity of the horizontal diameter

$$f = \frac{wr}{t} \pm \frac{\frac{1}{16} Wd}{\frac{1}{6} t^2} \dots\dots\dots (17)$$

At any point C (Fig. 7) the expression for the stresses may be written

$$f = \frac{wr \cos^2 \phi}{t} - \frac{qwr \sin^2 \phi}{t} \pm \frac{M}{\frac{1}{6} t^2} \dots\dots\dots (18)$$

These formulas are directly applicable to homogeneous elastic rings in which the modulus of elasticity of the material remains constant. These conditions are not strictly true for rings made of cast iron or of concrete or reinforced concrete. However, they may be applied without any great error to cast-iron rings and plain concrete rings at the breaking loads, if the modulus of rupture of the materials obtained under the same condition of thickness and loading be substituted for the maximum tensile stress f .

For a ring made of reinforced concrete the conditions differ somewhat from the foregoing. For ordinary cases it will be not far from the truth to equate the bending moment determined as above and the resisting moment of the reinforced concrete section. As the amount of reinforcement is usually lower than that in which the circular beam would fail by compression in the concrete, we may, without material error, take for the resisting moment of the reinforced concrete section the value .87 Aft , where t is the distance from the compression face to

the center of the steel reinforcement, A is the area of the cross-section of the reinforcement for a unit of length of ring, and f is the tensile unit-stress in the steel due to the bending moment. To equate the bending moment determined as before to this resisting moment is not exactly correct, since among other reasons the neutral axis does not come at the center of the thickness of the ring (which is the point about which the bending moments were taken), and since the elastic curve is not the same as in a ring of homogeneous material, and hence the distribution and amounts of the bending moments will not be exactly the same. However, the use of the bending moments determined for homogeneous rings is the nearest approximation we have, and is not seriously in error. At sections where thrust occurs, as at A, (Fig. 3), the tension in the steel determined as above will be reduced by the resisting compressive stresses there set up. The amount of the tension in the steel at the point A may be calculated by the formula

$$f' = f - \frac{\frac{1}{2} n T}{t (1 + np)} \dots\dots\dots (19)$$

which is applicable for both concentrated and distributed loads. In this formula f is the tensile stress in the steel due to the bending moment (as calculated by equating $.87Aft$ to the bending moment at the section considered), p is the ratio of the area of reinforcement for a unit length of beam or ring to the distance between the center of the steel and the compression face of the concrete, T is the thrust or pressure against the face of the section, and n is the ratio of the moduli of elasticity of steel and concrete, which, for purposes of this calculation, may be taken as 15. At the extremity of the horizontal diameter the thrust is $\frac{1}{2} W$. At the crown it is zero for vertical loading, and for both concentrated and distributed load the greatest tensile stress is found at this section.

9. *Horizontal and Vertical Deflections of Thin Elastic Ring.*—When a thin elastic ring is loaded with a symmetrical vertical load, the vertical deflection or change in vertical diameter may be determined from the expression $\int x d\theta$, where x is the abscissa with respect to the vertical diameter of the ring and θ is the angle which the normal section at any point has moved through due to the flexural distortion. This θ is the same as the angle used in the analysis on p. 6 for determining the bending moment for a concentrated load. Similarly, $\int y d\theta$ is the expression for the horizontal movement, y being the ordinate with

respect to the vertical diameter. It should be stated that these expressions neglect the change in shape due to the tangential thrust, but the amount of this change is slight. In this analysis the effect of the direct compression caused by the load is neglected, but this also is slight and for the dimensions discussed will have a very slight effect.

Space will not be taken here to derive these formulas by rigid analysis. The following may be helpful to the reader, however, in seeing the reasonableness of the expressions. In Fig. 7, the element of arc ds is subtended by the element of angle $d\phi$, only the center line of the ring being shown. θ (not indicated on this diagram) represents the change in direction of any normal section of the ring from its original direction, and any element of arc ds , as the one at C, will change direction by this angle θ . However, independently of this, each element will, by its own flexibility, change direction with respect to the ad-

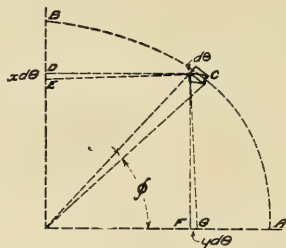


FIG. 7. DEFLECTION OF RINGS.

jacent element by an angle $d\theta$. By Fig. 7 it is seen that the effect of this change in direction is to throw the point at the end of the line CD to E, a distance $x d\theta$. The effect is the same as to cause a movement of the origin a distance $x d\theta$ upward, or what is the same thing, the point C must, by this action, be moved downward $x d\theta$, and this is the measure of the part of the movement of the element due to its own flexure. Of course by the methods of limits the seeming approximations of the diagram vanish and $\int x d\theta$ gives the total movement of the quadrant of the ring due to flexure.

Having the expressions $\int x d\theta$ and $\int y d\theta$, the next step is to substitute for $d\theta$ its value found from equations (2) and (3). The value of the variable bending moment M from equations (1) and (6) may then be substituted. Integration will be facilitated by expressing x and y in terms of r and ϕ . Integration may then be taken between $\phi = 0$

and $\phi = \frac{1}{2}\pi$. To get the change in diameter (two sides of the axis) the factor 2 must be introduced. The notation used will be that used in the analysis of bending moments.

For a concentrated load Q , we shall then have for the change in the vertical diameter

$$\begin{aligned}\triangle y &= 2 \int x d\theta = 2 \int \frac{M_x ds}{EI} = 2 \int \frac{Mr^2 \cos \phi \, d\phi}{EI} \\ &= \frac{2Qr^3}{EI} \int_0^{\frac{1}{2}\pi} (0.318 \cos \phi \, d\phi - \frac{1}{2} \cos^2 \phi \, d\phi) \\ &= -0.15 \frac{Qr^3}{EI} \dots\dots\dots (20)\end{aligned}$$

and for the change in the horizontal diameter

$$\begin{aligned}\triangle x &= 2 \int y d\theta = 2 \int \frac{M_y ds}{EI} = 2 \int \frac{Mr^2 \sin \phi \, d\phi}{EI} \\ &= \frac{2Qr^3}{EI} \int_0^{\frac{1}{2}\pi} (0.318 \sin \phi \, d\phi - \frac{1}{2} \cos \phi \sin \phi \, d\phi) \\ &= 0.136 \frac{Qr^3}{EI} \dots\dots\dots (21)\end{aligned}$$

For a vertical load W distributed uniformly over the horizontal section we shall have for the change in the vertical diameter

$$\begin{aligned}\triangle y &= 2 \int x d\theta = 2 \int \frac{M_x ds}{EI} = 2 \int \frac{Mr^3 \cos \phi \, d\phi}{EI} \\ &= \frac{1}{2} \frac{wr^4}{EI} \int_0^{\frac{1}{2}\pi} (\cos \phi \, d\phi - 2 \cos^3 \phi \, d\phi) \\ &= -\frac{1}{6} \frac{wr^4}{EI} = -\frac{1}{96} \frac{Wd^3}{EI} \dots\dots\dots (22)\end{aligned}$$

and for the change in the horizontal diameter

$$\begin{aligned}\triangle x &= 2 \int y d\theta = 2 \int \frac{M_y ds}{EI} = 2 \int \frac{Mr^3 \sin \phi \, d\phi}{EI} \\ &= \frac{1}{2} \frac{wr^4}{EI} \int_0^{\frac{1}{2}\pi} (\sin \phi \, d\phi - 2 \cos^2 \phi \sin \phi \, d\phi)\end{aligned}$$

$$= \frac{1}{6} \frac{wr^4}{EI} = \frac{1}{96} \frac{Wd^3}{EI} \dots\dots\dots (23)$$

For a combined distributed horizontal and vertical load, a similar treatment gives for the change in the vertical diameter

$$\Delta y = - \frac{1}{96} (1 - q) \frac{Wd^3}{EI} \dots\dots\dots (24)$$

and for the change in the horizontal diameter

$$\Delta x = \frac{1}{96} (1 - q) \frac{Wd^3}{EI} \dots\dots\dots (25)$$

where q is the ratio of the intensities of the vertical and horizontal loads.

These equations apply to rings of constant section and of constant modulus of elasticity. Although the modulus of elasticity of cast iron is not a constant, the formulas may be used with cast-iron rings, somewhat empirically. Reinforced concrete rings have a varied moment of inertia, due to the varying position of the reinforcement and to the effect of the tensile strength of the concrete, and the equations here given are not applicable.

10. *Modification for Thick Rings.*—In case the thickness of the ring is large in comparison with the diameter, (Fig. 8), the assumptions of the ordinary theory of flexure do not hold. The length of the inner fiber is less than that of the outer fiber, a condition which mod-

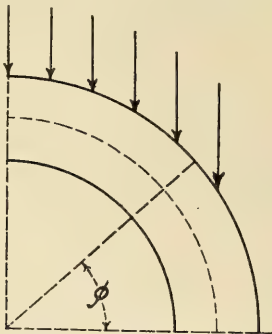


FIG. 8. THICK RING.

ifies the analysis considerably. As a result, it is found that the neutral axis is not at the center of the rectangular section, and a new set of formulas must be derived. The analysis of this condition will not be made here, since it is quite complicated and since, fortunately, for

the thickness used in ordinary construction the errors involved in using the common assumptions will be relatively small.

In the thesis of W. A. Slater on "Stresses in Concrete Rings Due to External Pressure," heretofore referred to, a thorough analysis is made of the stresses in thick rings. The treatment follows the method of Bach for curved beams. The analysis and the resulting equations are so long and complicated that the space available will not permit their publication in this bulletin. Table 1 gives the ratio of the stresses in a ring by formulas for curved beams to the results obtained by the formulas here given for several thicknesses of wall. The thickness of wall is given in terms of the mean diameter of the ring. The stresses obtained by the use of equations (10), (11), (13), and (14) multiplied by the ratios given in the table will be the amounts of stresses found by the formulas for curved beams. It is seen that the difference by the two methods is not large for the thinner rings. The moments in all cases were calculated on the assumption that the load was distributed over the mean diameter of pipe instead of over the exterior diameter. This approximation of itself will give a considerable error for thick

TABLE 1.

RATIO OF STRESSES.

The stresses by the formulas for curved beams may be found by multiplying the results obtained with equations (10), (11), (13), and (14) by the ratios given in the table.

Ratio of Thickness of Wall to Mean Diameter.		.05	.10	.15	.20	.25
Concentrated load	Top and bottom points					
	Tension	1.03	1.08	1.12	1.16	1.22
	Compression	0.97	0.94	0.92	0.89	0.87
	Side points					
	Tension	0.97	0.93	0.91	0.89	0.86
	Compression	1.02	1.07	1.12	1.15	1.21
Uniformly distributed load	Top and bottom points					
	Tension	1.03	1.08	1.12	1.16	1.22
	Compression	0.97	0.94	0.92	0.89	0.87
	Side points					
	Tension	0.98	0.93	0.89	0.85	0.81
	Compression	1.03	1.07	1.10	1.13	1.16

rings. Of course, for thicker rings the effect of the difference in length of inner and outer fiber increases rapidly, and the value of the stresses obtained by the method here employed is soon in material error. However, it must be borne in mind that, as the reinforced concrete pipe has varying stiffness around the circumference depending upon whether the concrete has failed in tension and upon the position of the steel in the section, the values of the bending moments given by equations (5) and (7) are only approximate and refinements in the calculation of the resisting moment are not warranted.

11. *Conditions of Bedding and Loading Found in Practice.*—The foregoing discussion assumes certain definite conditions of loading. These are useful in establishing definite formulas which may be used as a basis for calculations. It is not to be expected that these conditions represent accurately the condition of bedding and loading to be found in practice. It is then desirable that the nature and extent of possible or probable variations from these assumed conditions be discussed and the effects of such a divergence considered. The following are suggestions of variations; the engineer will easily extend the discussion by numerous examples taken from his own experience.

If the layer of earth immediately under the pipe is hard or uneven, or if the bedding of the pipe at either side is soft material or not well tamped as indicated in Fig. 9 (a), the main bearing of the pipe may be along an element at the bottom and the result is in effect concentrated loading. The result is to greatly increase the bending moment developed and hence the tendency of the pipe to fail. This condition may be aggravated in the case of a pipe with a stiff hub or bell where settlement may bring an unusual proportion of the bearing at the bell and the distribution of the pressure be far from the assumed condition. In bedding the pipe in hard ground it is much better to form the trench so that the pipe will surely be free along the bottom element, even after settlement occurs, and so that the bearing pressures may tend to concentrate at points say under the one-third points of the horizontal diameter (or even the outer quarter points). This will reduce the bending moments developed in the ring, as has been suggested on page 8.

In case the pipe is bedded in loose material, the effect of the settlement will be to compress the earth immediately under the bottom of the pipe more completely than will be the effect at one side, as indicated in Fig. 9(b), with the result that the pressure will not be uniformly distributed horizontally. Similarly, in a sewer trench, if loose

material is left at the sides and the material at the extremity of the

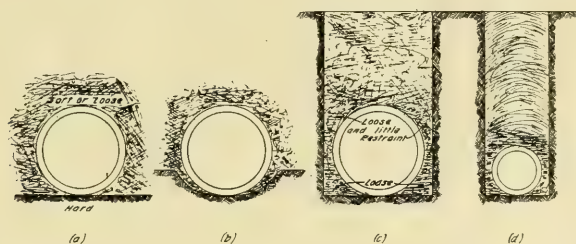


FIG. 9. CONDITIONS OF LOADING.

horizontal diameter is loose and offers little restraint, as indicated in Fig. 9(b), the pressure on the earth will not be distributed horizontally and the amount of bending moment will be materially different from that where careful bedding and tamping give an even distribution of bearing pressure over the bottom of the sewer.

In case of a small sewer in a deep trench, the load upon the sewer may be materially less than the weight of the earth above, as in the

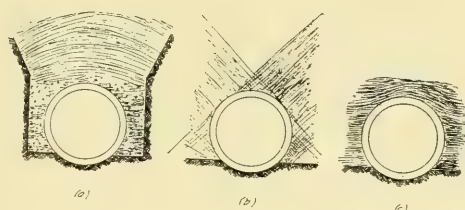


FIG. 10. CONDITIONS OF LOADING.

case shown in Fig. 9(c), where the earth forms a hard compact mass and is held by pressure and friction against the sides of the trench.

In case a culvert pipe is laid in an ordinary embankment by cutting down the sides slopingly as shown in Fig. 10(a), it is evident that the load which comes upon the pipe will be materially less than the weight of the earth immediately above it. If a culvert pipe replaces a trestle and the filling is allowed to run down the slope as shown in Fig. 10(b), the direction and amount of the pressure against the pipe will differ considerably from that which obtains in a trench or in the case of a level filling shown in Fig. 10(c). It is possible in the latter case that the smaller amount of settlement of the earth directly over the culvert pipe, due to the greater depth of earth on the adjacent sections, may

allow a greater proportion of the load to rest upon the culvert pipe than would ordinarily be assumed.

Attention should be called to the fact that the distribution of the pressure by means of earth under and over a ring assumes that the earth is compressed in somewhat the same way as when other material of construction is given compression. Unless the earth has elasticity, the distribution of pressure cannot occur. To secure the uniform distribution assumed the ring itself must give enough to allow for the movement of the earth which takes place under pressure. This is especially true with reference to the presence and utilization of lateral restraint, and a ring which does not give laterally, as for example a plain concrete ring, will not develop lateral pressure in the adjoining earth under ordinary conditions of moisture and filling to any great extent. As the conditions of earth and moisture produce mobility and approach hydrostatic conditions, the necessity for this elasticity and movement do not exist, but here the lateral pressure approaches the vertical pressure in amount and the bending moments become relatively smaller.

The discussion is sufficiently extended to indicate the importance of care in bedding culvert pipe and sewers and in filling over them, and to indicate the great difference in the amount of bending moment developed with different conditions of bedding and filling. Where there is any question of needed strength, it will be money well expended to use care and precaution in bedding the pipe and in filling around and over it. I am convinced that a little extra expense will add considerable stability, life, strength, and safety to such structures, far out of proportion to the added cost. It is possible that under careful conditions of laying, lighter structures may be used with a saving in the cost of construction.

III. MATERIALS, TEST PIECES, AND METHOD OF TESTING.

12. *Cast-Iron Pipe and Rings.*—Nine cast-iron culvert pipes were tested, four being of 36-in. inside diameter and five of 48-in. inside diameter. Four pipes were furnished by the Atchison, Topeka & Santa Fe Ry., two each by the Chicago, Milwaukee & St. Paul Ry., and the Chicago, Rock Island & Pacific Ry., and one by the Illinois Central R. R. Both light-weight pipe and medium-weight pipe were tested. With the exception of No. 990 there was little variation in the thickness in different points of the pipes, the uniformity being greater than is found in the usual run of pipe. In Pipe No. 990 the thickness varied from 0.75

TABLE 2.

TESTS OF CROSS-BREAKING SPECIMENS FROM CAST-IRON PIPES.

From Pipe No.	Span inches	Cross-section in. x in.	Modulus of Rupture lb. per sq. in.	Modulus of Elasticity lb. per sq. in.
991	24	1.995 x 1.355	41 000	10 700 000
991	24	1.984 x 1.358	37 500	11 500 000
991	24	1.990 x 1.350	38 700	12 800 000
991	24	1.995 x 1.385	32 300	11 200 000
992	24	1.980 x 1.298	42 000	12 600 000
992	24	1.985 x 1.293	43 100	12 600 000
992	22	1.990 x 1.340	39 500	10 900 000
992	22	1.995 x 1.236	41 600	12 200 000
993	24	1.980 x 1.500	39 500	11 000 000
993	24	2.000 x 1.460	43 500	12 000 000
993	24	1.960 x 1.440	42 800	12 850 000
993	24	2.000 x 1.440	43 300	13 200 000
993	22	1.970 x 1.472	38 600	11 000 000
993	22	1.985 x 1.481	36 300	10 600 000
994	24	1.987 x 1.575	35 600	9 900 000
994	24	1.987 x 1.552	33 400	10 400 000
994	24	2.019 x 1.450	31 300	10 900 000
994	24	1.976 x 1.442	33 200	11 100 000
995	24	2.015 x 0.845	30 500	10 500 000
995	24	1.980 x 0.851	34 900	10 400 000
995	24	1.990 x 1.000	30 400	9 940 000
995	24	1.979 x 1.000	33 500	10 000 000
995	22	1.991 x 1.100	35 400	13 200 000
995	22	1.980 x 1.096	36 100	13 400 000
996	24	1.996 x 1.180	33 400	11 200 000
996	24	1.986 x 1.205	32 800	11 300 000
996	22	1.980 x 1.248	31 800	12 600 000
996	22	1.988 x 1.248	31 500	11 800 000
996	24	1.984 x 1.229	31 300	12 300 000
996	24	1.988 x 1.223	30 100	12 500 000
997	24	1.979 x 0.962	35 800	14 200 000
997	24	1.993 x 0.948	37 600	14 500 000
997	22	1.980 x 0.787	30 900	13 800 000
997	22	1.993 x 0.830	28 600	13 000 000
997	24	1.996 x 0.695	34 800	13 100 000
997	24	1.976 x 0.687	35 900	13 800 000
998	24	1.978 x 1.170	38 500	12 100 000
998	24	1.975 x 1.175	39 500	12 100 000
998	24	2.005 x 1.300	40 300	11 800 000
998	24	2.000 x 1.280	39 200	12 700 000
998	24	1.987 x 1.361	33 100	11 900 000
998	24	1.997 x 1.318	33 800	14 000 000
998	22	1.981 x 1.426	38 200	12 800 000
998	22	1.997 x 1.458	38 900	11 500 000

to 1.75 in. One pipe was 6 ft. 8 in. long, and the others ranged from 8 ft. 0 in. to 8 ft. 5 in. over all after the rings for use in the auxiliary tests had been cut from them. Test pieces, 2 inches wide and about 24 inches

TABLE 3.
CAST-IRON CULVERT PIPE AND RINGS.

No.	Internal Diameter inches	Thickness		Length Over All inches	Furnished by
		Range inches	Average inches		
Cast-Iron Pipes.					
990	48		1.25	80	I. C. R. R.
991	48		1.25	99	C. M. & St. P. Ry.
992	48		1.25	99	C. M. & St. P. Ry.
993	48		1.50	101	C. R. I. & P. Ry.
994	48		1.50	96	C. R. I. & P. Ry.
995	36		1.00	96	A. T. & S. F. Ry.
996	36		1.25	96	A. T. & S. F. Ry.
997	36		1 00	96	A. T. & S. F. Ry.
998	36		1.25	96	A. T. & S. F. Ry.
Cast-Iron Rings.					
991A	48		1.4	24	C. M. & St. P. Ry.
991B	48		1.25	24	C. M. & St. P. Ry.
992A	48			24	C. M. & St. P. Ry.
992B	48	1.31—1.25	1.28	23	C. M. & St. P. Ry.
993A	48	1.47—1.35	1.43	24	C. R. I. & P. Ry.
993B	48	1.50—1.34	1.42	24	C. R. I. & P. Ry.
994B	48	1.63—1.53	1.58	24	C. R. I. & P. Ry.
994C	48		1.6	24	C. R. I. & P. Ry.
995A	36	1.13—0.90	1.02	24	A. T. & S. F. Ry.
995B	36		1.1	24	A. T. & S. F. Ry.
996A	36		1.44	24	A. T. & S. F. Ry.
996B	36	1.50—1.27	1.42	24	A. T. & S. F. Ry.
997A	36		1.00	24	A. T. & S. F. Ry.
997B	36	0.98—0.92	0.91	24	A. T. & S. F. Ry.
998A	36		1.25	24	A. T. & S. F. Ry.
998B	36	1.31—1.05	1.19	24	A. T. & S. F. Ry.

long, were afterward cut from the rings and pipes and tested in cross breaking. The results are given in Table 2. The quality of the cast iron in the pipes was very good. Data on the pipes and rings are given in Table 3, p. 24.

13. *Reinforced Concrete Culvert Pipe.*—Five reinforced concrete culvert pipes were furnished by the Chicago, Burlington & Quincy R. R.

These pipes were designed by Mr. C. H. Cartlidge, Bridge Engineer. They were made at Montgomery, Ill. Owl portland cement was used. The gravel came from a pit at Montgomery. A mechanical analysis of the gravel is given in Table 4, page 25. In two pipes, the reinforcement was $\frac{1}{4}$ -in. corrugated bars, placed as shown in Fig. 11, and $\frac{1}{2}$ -in. corrugated bars were used in one pipe, No. 982. One pipe, No. 988,

TABLE 4.

MECHANICAL ANALYSIS OF GRAVEL USED IN REINFORCED
CONCRETE CULVERT PIPE.

Sieve No.	Per cent passing			
	Test No. 1	Test No. 2	Test No. 3	Average
$\frac{3}{4}$	90.5	93.8	91.0	91.8
$\frac{1}{2}$	77.1	81.5	78.1	78.9
$\frac{3}{8}$	63.0	68.4	65.8	65.7
$\frac{1}{4}$	50.7	58.0	55.0	54.9
5	35.4	43.7	40.2	39.8
10	33.4	41.5	38.3	37.7
12	33.1	41.1	37.8	37.3
16	32.0	40.4	37.2	36.5
18	31.7	39.4	36.1	35.7
30	29.6	36.1	32.9	32.9
40	23.4	28.4	26.0	25.9
50	9.0	10.5	10.1	9.9
74	5.1	6.4	6.0	5.8
150	1.3	1.9	1.7	1.6

was reinforced with "Clinton Wire Mesh," No. 3 wire being used, and the fifth pipe was reinforced with fence wire laid in the center of gravity of the concrete. The steel which was placed lengthwise of the pipe is not considered in figuring the reinforcement. To allow the circumferential reinforcement to be made circular in shape, the pipes were made with the vertical diameter four inches longer than the horizontal diameter. This brought the reinforcement at the points where tension would be present in the loaded pipe. Four of these pipes had a nominal inside horizontal diameter of 48 in. and the other, a nominal inside horizontal diameter of 36 in. This horizontal diameter will be used in referring to the pipe, the vertical diameter being 4 in. longer. The barrel of the pipe was 4 in. thick. The bell extended 4 in. beyond the barrel and had the same thickness, its inside diameter being 1 in. greater than the outside diameter of the barrel.

The forms were of wood lined with galvanized iron and were placed vertically with the bell up. These forms were removed when the concrete was from two to four days old and the pipes were stored in the open air until after shipment to the University, where they were stored in the testing laboratory. Pipe No. 982 was made in cold weather and was heated by steam coils to aid the curing of the concrete. Examination indicated that the quality of the concrete was somewhat injured thereby. All the pipes had very good surface and, when broken up, appearances indicated a very good quality of concrete.

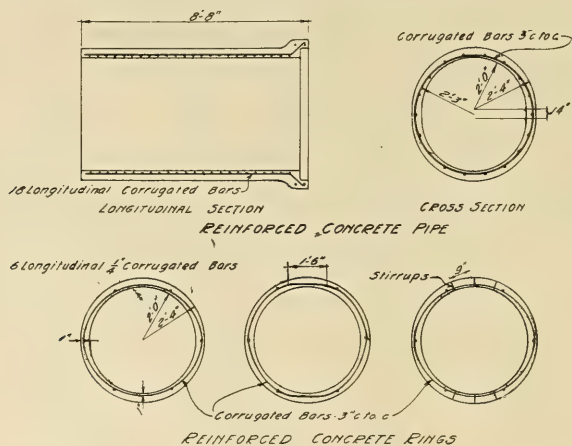


FIG. 11. DISPOSITION OF REINFORCEMENT IN THE REINFORCED CONCRETE PIPE AND RINGS.

Two pipes furnished were of the type known as Jackson pipe, which one of the railroad companies had in stock. They were made by the Reinforced Concrete Pipe Co., Jackson, Mich. The pipe was 48 in. in internal diameter with walls 5 in. thick. The length was 3 ft. and two sections were placed together in the test. This pipe has steel bands at the middle of the thickness. It is not expected that the reinforcement is placed most advantageously, but the steel serves to strengthen the pipe in handling and obviates sudden failure. It is here treated with the plain concrete rings.

14. *Concrete and Reinforced Concrete Rings.*—The concrete and reinforced concrete rings were made in the laboratory at the University. The sand came from near the Wabash River at Attica, Indiana. It was of good quality and contained from 28% to 30% voids, as determined by standard methods. A mechanical analysis of this sand is

given in Table 5, p. 27. The stone was Kankakee limestone which showed about 50% voids. In Table 6, p. 27, is given a mechanical analysis of this stone. Universal portland cement, furnished by the manufacturers, was used. Table 7, p. 28, gives the results of tensile

TABLE 5.
MECHANICAL ANALYSIS OF SAND.

Sieve No.	Separation Size inches	Per cent passing	
		Test No. 1	Test No. 2
5	.174	99.8	99.5
10		80.8	80.0
12	.067	73.8	71.2
16		67.4	63.4
18	.043	52.9	48.2
30	.027	26.2	25.9
40	.019	15.0	16.1
50	.013	5.4	7.6
74	.009	1.9	3.6

tests of briquettes made from this cement according to standard methods. The concrete, which was made rather wet, was thoroughly mixed by hand in the usual manner, the mixture being 1-2-4 in all cases.

TABLE 6.
MECHANICAL ANALYSIS OF STONE.

Size of Mesh inches	Per cent passing	
	Test No. 1	Test No. 2
1.25	100.0	100.0
1.00	95.0	95.0
.50	40.0	29.0
.25	7.5	2.0
.125	2.5	1.0

The steel used in the reinforced rings was $\frac{1}{4}$ -in. mild steel corrugated bars. Table 8, p. 28 shows the results of tension tests of this steel, each result being the average of two to four tests.

The rings were all circular in section, 4 ft. inside diameter, and 2 ft. long, the reinforcement being placed as shown in Fig. 11. The

TABLE 7.
TENSILE STRENGTH OF CEMENT.

Ref. No.	Ultimate Strength, lb. per sq. in.			
	Age 7 days		Age 28 days	
	Neat	1-3	Neat	1-3
1	410	187	680	370
2	470	200	670	330
3	360	120	560	360
4	405	145	570	290
5	320	195	600	295
6	310	180	620	310
Av.	379	171	617	324

reinforced rings, of which there were 16, were 4 in. thick, while the 8 plain concrete rings were made 6 in. thick.

In most of the rings the steel was shaped somewhat like an ellipse in order to bring the reinforcement near the tension side at the quarter

TABLE 8.
TENSION TESTS OF STEEL USED IN REINFORCED CONCRETE RINGS.

Specimen taken from Ring No.	Nominal Size inches	Area sq. in.	Per cent Elongation in 8 inches	Yield Point lb. per sq. in.	Ultimate Strength lb. per sq. in.
921	1/4-in. corr.	0.06	21	45 550	62 700
922	"	0.06	22	47 000	64 550
923	"	0.06	22	47 700	68 450
926	"	0.06	23	46 600	66 000
927	"	0.06	22	47 400	69 000
928	"	0.06	23	47 150	68 400
931	"	0.06	23	44 900	63 700
932	"	0.06	23	46 900	65 900
933	"	0.06	23.5	46 400	66 200
934	"	0.06	24	45 300	63 250
951	"	0.06	22	44 300	66 200
952	"	0.06	24	47 400	66 800
953	"	0.06	22	45 650	65 700
971	"	0.06	22	44 550	64 950
972	"	0.06	22	47 900	67 000
977	"	0.06	24	45 100	64 550

points. In No. 934, 971, and 972 the bars were flattened for a distance of 18 inches at both top and bottom of the ring, and in No. 976 and 977 three crimped pieces of steel were placed at top and bottom across the ring. These were spaced 9 in. apart and served the same purpose as stirrups in a beam.

The rings were made on the floor of the laboratory, the forms being removed 7 days after making, after which the rings were wet twice daily with a hose until tested. Fig. 12 shows a group of these rings. From

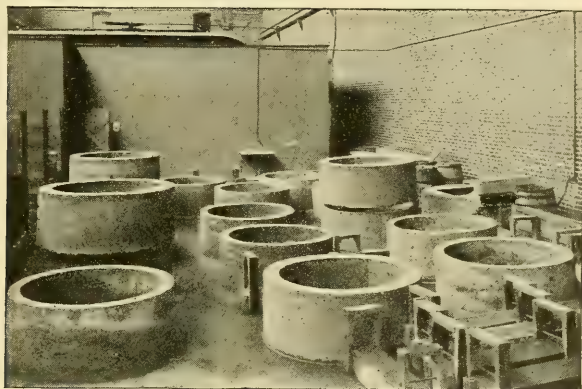


FIG. 12. VIEW SHOWING CONCRETE AND REINFORCED CONCRETE RINGS.

each batch of concrete from which a ring was made there was taken enough concrete to make an unreinforced beam 6 in. x 8 in. x 40 in. These were tested in flexure when about 30 days old as a check on the modulus of rupture of the concrete which went into the ring. They were tested with 3 ft. span with load at the one-third points. The average modulus of rupture, as obtained from the tests of 22 of these beams, was 195 lb. per sq. in.

15. *Testing Machine for Distributed Load.*—The machine used for testing the culvert pipe is shown in Fig. 13. The photograph given in Fig. 14 will aid in giving an idea of the apparatus used. The floor of the machine rested on two I-beams. A layer of sand was spread over this floor. On this layer of sand was placed the pipe to be tested. The sides of the box were built up with bridge timbers and held firmly laterally by heavy rods. Sand was put around and over the pipe. The sand was leveled off at the top and the saddle built. Two I-beams, similar to those below the pipe, were placed on the saddle and at the

The rings which were tested with distributed load were set up in a similar way except that the box and saddle were only long enough to accommodate a 2-ft. length of pipe. The box was set up on the bed of the 600 000-lb. Riehle testing machine and the load was applied by running the head down on the saddle. Readings of the horizontal and vertical diameters were taken at the center of the ring, the load being applied in increments of 1000 lb.

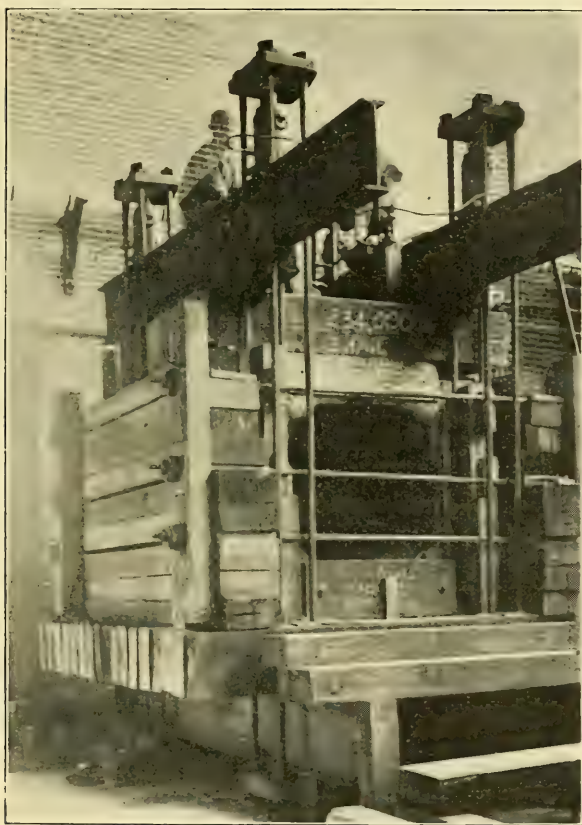


FIG. 14. VIEW OF HYDRAULIC JACK TESTING MACHINE.

All the testing under concentrated load was done in the 600 000-lb. machine. A layer of plaster of paris about $\frac{1}{4}$ -in. thick was spread on a strip of building paper lying on the bed of the machine. The ring was put in place so that the bottom element rested in the plaster of paris. A similar layer of plaster was put on the top of the ring and a

cast-iron bar 3 in. wide by 2 in. thick by 2 ft. long was carefully centered in the plaster over the top element. A second bar of the same dimensions was afterward placed on top of this to give greater stiffness. The head of the machine was then run down and the load applied through a spherical bearing block. In the tests of cast-iron rings cushions of wood replaced the plaster of paris.

IV. EXPERIMENTAL DATA WITH CAST-IRON PIPE AND RINGS.

17. *Data of Tests.*—Table 3 (p. 24) gives dimensions of the cast-iron culvert pipe and rings and the name of the railroad company which furnished the pipe. The suffix A denotes the ring cut from the spigot end and the suffix B the ring next to it. In all cases the remainder of the pipe as tested contained the bell of the original pipe (given in Table 3, under the heading of Cast-Iron Pipes). Table 9 (p. 32) gives the results of the concentrated-load tests and Table 10 (p. 42) the results of the distributed-load tests.

In Fig. 16 to 21 (p. 34 to 41) are given diagrams showing the amount of the vertical and horizontal deflections of the pipe, at both the spigot end and the bell end, which were produced under the load per lineal foot of pipe or ring indicated by the vertical scale. These diagrams will be discussed for the two methods of loading.

TABLE 9.
CAST-IRON RINGS. CONCENTRATED LOAD.

No.	Breaking Load lb. per lin. ft.	Modulus of Rupture lb. per sq. in.	Modulus of Elasticity lb. per sq. in.	Remarks
991A	16 250	32 600	11 000 000	Failed at top.
991B	10 300	26 000	12 700 000	Failed at bottom.
992B	11 720	27 300	9 700 000	Failed at top.
993B	12 850	25 200	7 780 000	Failed at bottom.
994B	17 400	27 600	8 300 000	do.
994C	16 500	25 500	8 600 000	Failed at top.
995A	9 650	27 500	10 200 000	Failed at bottom.
995B	13 500	24 000	14 300 000	Failed at top.
996A	17 500	33 300	11 840 000	Failed at bottom.
996B	18 500	26 800	8 800 000	do.
997A	6 950	20 600	6 500 000	Failed at top.
997B	6 650	21 800	10 900 000	Failed at bottom.
998A	17 500	33 300	12 400 000	Failed at top.
998B	13 100	23 800	9 700 000	Failed at bottom.

18. *Concentrated-Load Tests.*—The cast-iron rings which were tested under a concentrated load gave phenomena similar to what would be expected from the analysis of the bending moments and resisting moments. As is shown on the diagram in Fig. 16, the amount

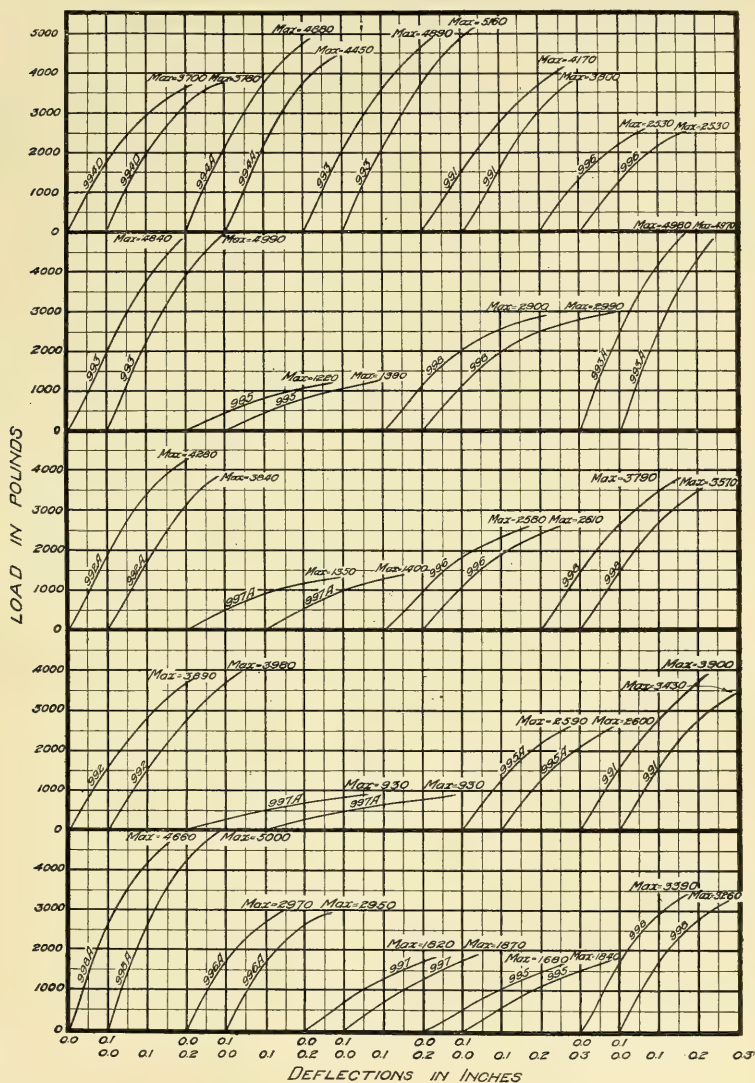


FIG. 15. LOAD-DEFLECTION DIAGRAMS FOR CAST-IRON TEST SPECIMENS.

of deflection is nearly proportional to the applied load. The value of the modulus of elasticity given in Table 9, calculated from equation (20) for loads and deflections near the breaking load, 10 200 000 lb. per sq. in., is somewhat smaller than the average value of the modulus of elasticity determined from the tests of rectangular specimens cut from the pipe given in Table 2. The latter are determined from the straight portion of the load-deflection diagrams given in Fig. 15. This is not unexpected for a material as variable in its elastic properties as cast iron and with such a difference in form as the ring and the straight rectangular beam. It seems that the value of the modulus of elasticity determined from the deflections of the rings may best be used in the investigation of the rings and pipes.

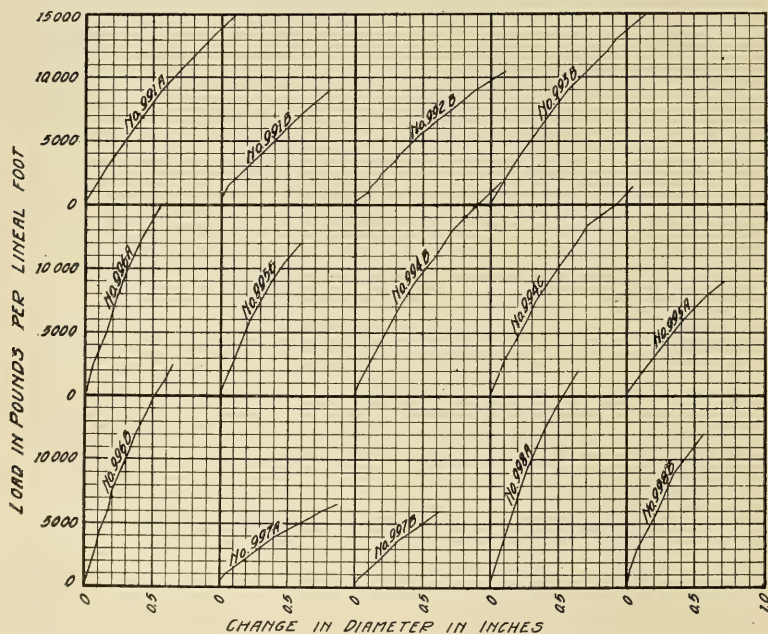


FIG. 16. LOAD-DEFLECTION DIAGRAMS FOR CAST-IRON RINGS. CONCENTRATED LOAD.

At the maximum load sustained the rings failed at either the top or the bottom, and the rupture extended over the entire length of the pipe in every case. This accords with the theoretical determination already made, it having been found that the bending moment at the top or bottom is about $\frac{1}{3}$ that at the extremities of the horizontal

diameter. After rupture occurred the machine was in some cases run on down to give greater deflections, but the load then sustained was much less than the breaking load and the rings finally broke again, this time at the ends of the horizontal diameter.

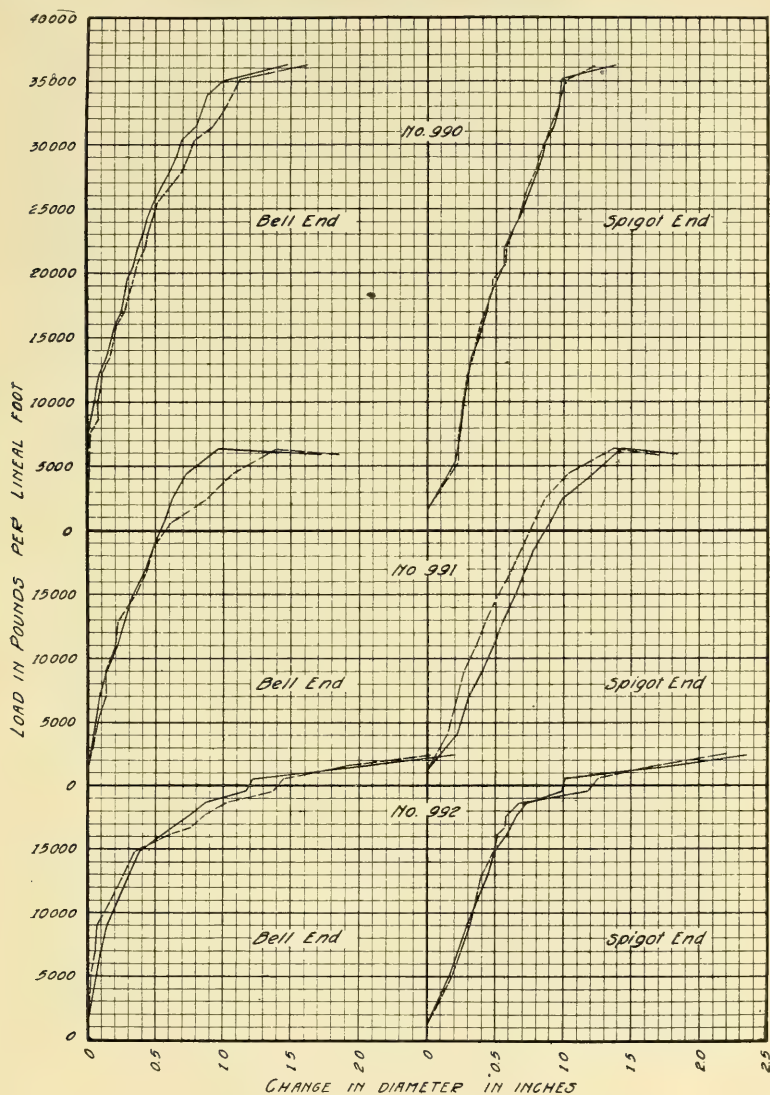


FIG. 17. LOAD-DEFLECTION DIAGRAMS FOR 48-IN. CAST-IRON PIPES.
DISTRIBUTED LOAD.

It will be seen that the average value of the modulus of rupture calculated by equation (10) and given in Table 9, is 27 000 lb. per sq. in. This is 25% less than the value of the modulus of rupture given

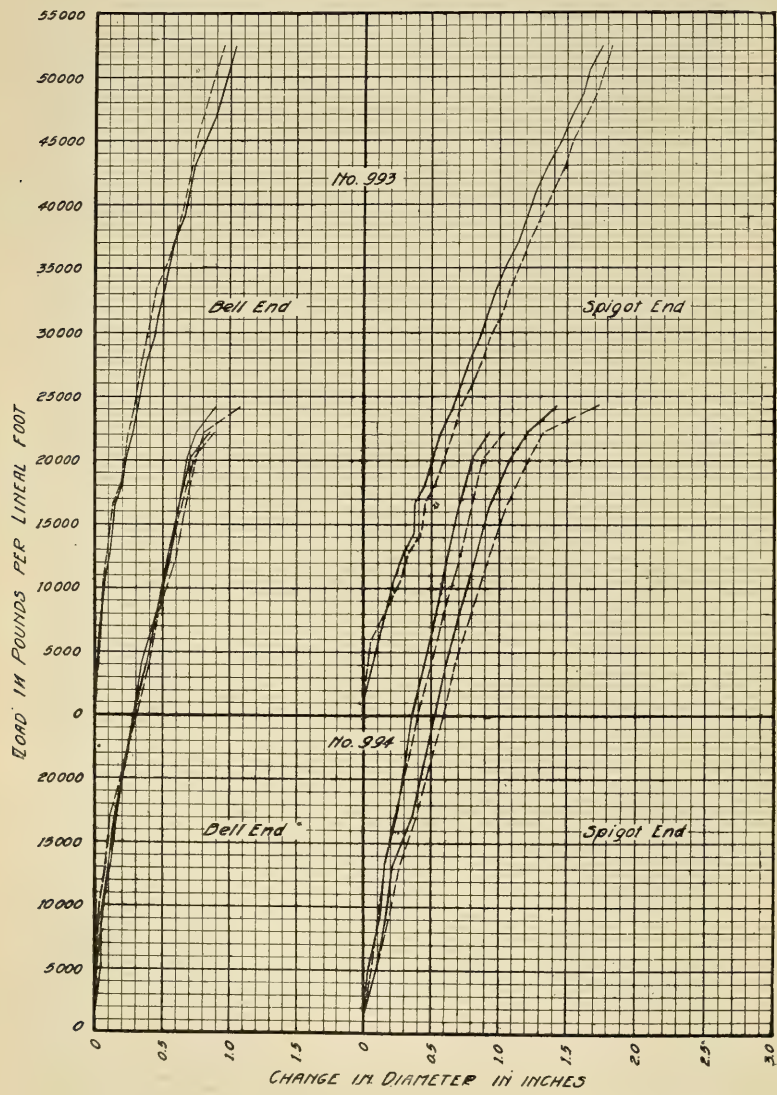


FIG. 18. LOAD-DEFLECTION DIAGRAMS FOR 48-IN. CAST-IRON PIPES.
DISTRIBUTED LOAD.

in Table 2 as determined by loading at the center the rectangular test specimens taken from the rings. The difference which exists in the distribution of the stresses by these two methods of testing and the dif-

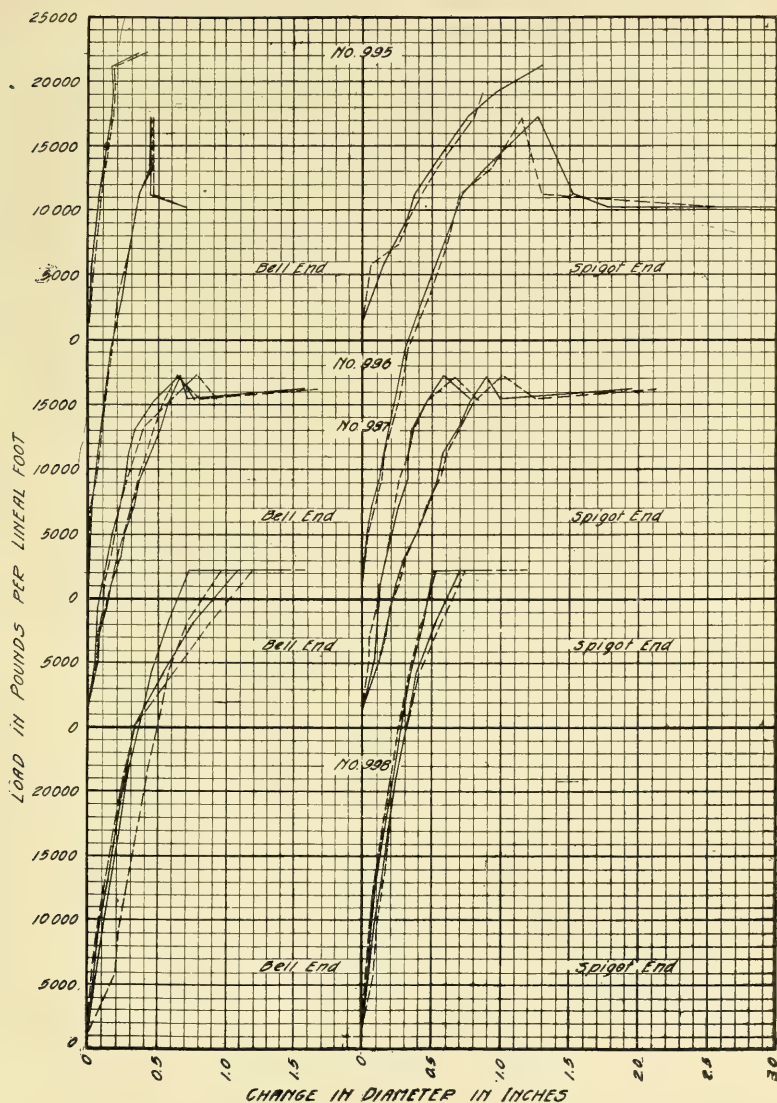


FIG. 19. LOAD-DEFLECTION DIAGRAMS FOR 36-IN. CAST-IRON PIPES.
DISTRIBUTED LOAD.

ferences in strength of the materials in the two directions will partly account for this difference. It seems probable that the results given in Table 9 are more nearly representative of the condition in the rings and pipes, and in the calculations for the pipes and rings under distributed load the value obtained in the concentrated tests of the corresponding ring has been used.

19. *Phenomena of the Distributed-Load Tests.*—In the distributed-load tests the pipe or ring was bedded in sand, the load was applied through a saddle resting on sand, and the sides of the test box

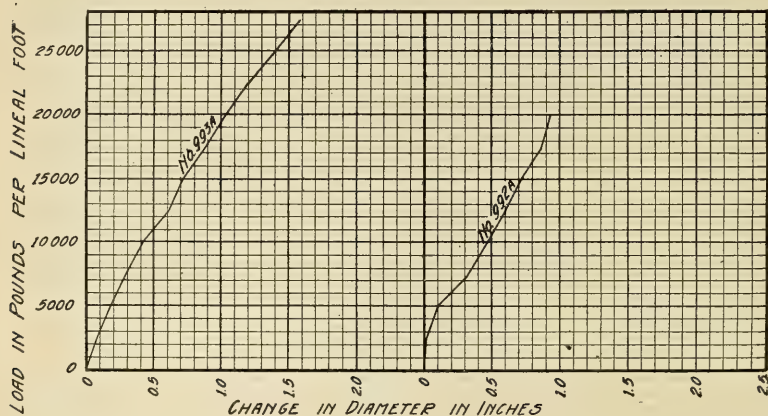


FIG. 20. LOAD-DEFLECTION DIAGRAMS FOR 48-IN. CAST-IRON RINGS.

were restrained, as described on p. 29. It is not probable that such a method of loading will give a uniform distribution of the load either longitudinally or transversely. It is evident from a study of results that the distribution of the load is uncertain and that the amount of the lateral restraint is also uncertain. While for the purposes of calculation and discussion the distribution of the pressure may be assumed to be uniform over a horizontal section, yet it is plain that this does not express at all accurately the manner in which the load was distributed. However, a uniform distribution of the load may be used as a basis of comparison of the discussion of the results. A study of the load-deflection diagrams given in Fig. 17 to 21, will show that the deflection at the spigot end generally began at once and increased nearly proportionally to the amount of the applied load, as is indicated by the approximation to a straight line. This is in accord with the analysis and with

the form of equation (22). At the bell end the deflections lagged behind and generally formed a curved diagram. It is evident that the great stiffness of the bell is the cause of this and that after the barrel of the pipe becomes distorted the effect is transmitted to the bell in varying proportions. At a load varying from 75% to 100% of the maximum a crack appeared in the top or bottom of the pipe except in No. 996 where it appeared at one side. Usually with an increase of load this crack extended toward the other end, and finally the pipe broke throughout the full length at or near the maximum load. This break generally occurred at the top or bottom of the pipe, and gave a loud report. The further operation of the testing apparatus gave increased deflections, sometimes without a material reduction in the load and sometimes at a much lower load, until the pipe broke at some other point in the section.

The general phenomena of the tests of the cast-iron pipes are given in the following notes.

No. 990. Diameter 48 in. Length 6 ft. 8 in. over all. Variable thickness, ranging from $\frac{3}{4}$ in. to $1\frac{3}{4}$ in. Heavy bell. Center of loading apparatus placed 12 in. from middle of pipe toward bell end. Crack appeared in barrel next to bell at a load of 26 700 lb. per lin. ft. At 27 900 lb. the crack extended from the bell 7 in. At 30 300 lb. the crack extended $1\frac{1}{2}$ in. into the bell and at 31 500 it extended entirely through the bell and 14 in. along the barrel. At 36 300 lb. a crack appeared on the north side next to the bell. At 37 500 lb. per lin. ft. complete failure occurred, the crack on north side extended from end to end, a crack in the south side appeared and extended from the spigot end to within 2 ft. of the bell and then through the bell, and a crack formed at the bottom. Test discontinued.

No. 991. Diameter 48 in. Length 8 ft. 3 in. over all. Light-weight pipe, 7000 lb. for a 12-ft. net length. Average thickness $1\frac{1}{4}$ in. Loaded 2 in. off center. At a load of 22 600 lb. per lin. ft. a crack appeared at the top and extended 3 ft. from the spigot end. At 24 500 lb. a crack appeared at the bottom and extended through the bell and 3 ft. into the barrel. At 26 500 lb. per lin. ft. complete failure occurred, the crack at the bottom extended the full length of the pipe and the crack at the top extended past the center. The load dropped to 26 000 lb. and cracks appeared at both north and south sides. Test discontinued.

No. 992. Diameter 48 in. Length 8 ft. 3 in. over all. Light-weight pipe, 6900 lb. for 12-ft. net length. Small bell. Average thickness 1.3 in. Loaded 2 in. off center. At a load of 16 300 lb. per lin. ft. cracks formed at the top and bottom and extended through the bell 8 in. into the barrel. At 17 700 lb. the bottom crack extended from end to end, and at 18 700 lb. the top crack extended likewise. At 21 600 lb. both sides cracked from end to end. The load dropped off, and finally rose again, reaching 22 600 lb. per lin. ft. Test discontinued.

No. 993. Diameter 48 in. Length 8 ft. 5 in. over all. Heavy-weight pipe, 8900 lb. for 12-ft. net length. Average thickness 1.42 in. Loaded $2\frac{1}{2}$ in. off center. No crack appeared until the maximum load of 54 600 lb. per lin. ft., when the pipe cracked from end to end at the top. The load dropped off to about 44 000 lb. per lin. ft., and the pipe almost immediately cracked from end to end at bottom and two sides without further application of the testing apparatus. Test discontinued.

No. 994. Diameter 48 in. Length 8 ft. net. Heavy-weight pipe, 8900 lb. for 12-ft. net length. Average thickness 1.58 in. This pipe was cut into two pieces and the end of one was loosely inserted in the bell of the other at the middle of the testing box. The piece containing the bell is denoted as the bell section, and the other the spigot section. The center of the loading apparatus was over the bell. The first crack appeared in the bell at the top and extended 2 ft. into the barrel at a load of 47 200 lb. per lin. ft. At the maximum load of 49 300 lb. per lin. ft. this crack extended the full length of the bell section. No crack appeared in the spigot section. Test discontinued.

No. 995. Diameter 36 in. Length 8 ft. 0 in. over all. Light-weight pipe, 4575 lb. for 12-ft. net length. Average thickness 1.02 in. Loaded $2\frac{1}{2}$ in. off center. At a load of 19 300 lb. per lin. ft. a crack appeared and extended along the top 6 ft. from the spigot end. This crack lengthened at succeeding loads and extended from end to end at the maximum load of 22 300 lb. per lin. ft. At this load a crack appeared at the bottom and extended from end to end. Cracks also appeared at the sides at the spigot end and instead of extending along an element of the cylinder they turned upward and joined the first crack at the top just inside the bell. Test discontinued.

No. 996. Diameter 36 in. Length 8 ft. 0 in. over all. Medium-weight pipe, 5600 lb. for a 12-ft. net length. Average thickness $1\frac{1}{4}$ in. Loaded $2\frac{1}{2}$ in. off center. The first crack appeared at one side at the spigot end at a load of 37 300 lb. per lin. ft., and extended 6 ft. The load dropped off and a crack at once appeared on the other side extending almost the entire length of the pipe. Upon increasing the deflections, at a load of 30 300 lb. a crack appeared at the top and extended from end to end, and with further deflection the one side crack extended through the bell and the other turned upward and joined the top crack just inside the bell. The crack at the bottom extended from end to end. Test discontinued.

No. 997. Diameter 36 in. Length 8 ft. 0 in. net. Light-weight pipe, 4300 lb. for a 12-ft. net length. Nominal thickness 1 in. This pipe was cut into two pieces and the end of one was loosely inserted in the bell at the middle of the testing box. The center of the loading apparatus was over the bell. The first crack appeared in the bell at the top at a load of 25 200 lb. per lin. ft. and was 4 in. long. At the maximum load, 27 300 lb. per lin. ft. this crack had lengthened to 20 inches. The load dropped to 25 500 lb., and with increased deflection, a crack formed in the bottom of the bell section passing through the bell 24 in. into the barrel. This was at once followed by the formation of a crack at the bottom in the spigot section extending from end to end. With further application of the testing apparatus the cracks at the top and bottom of the bell section extended from end to end at a load but little below the maximum. Finally both sections had cracked from end to end at all four quarter points. Test discontinued.

No. 998. Diameter 36 in. Length 8 ft. 0 in. net, cut in two sections, with the bell placed in the center as in No. 997. Medium-weight pipe, 5700 lb. for a 12-ft. net length. Nominal thickness of barrel $1\frac{1}{4}$ in. Center of load over bell. First crack appeared at both top and bottom of the bell section at a load of 29 200 lb. per lin. ft. and was 6 in. long, soon extending to 18 in. The bell section cracked end to end at the top and half of the length at the bottom at the maximum load, 37 200 lb. per lin. ft. On increasing the deflection at about the same load, the spigot section cracked from end to end at both top and bottom, and the bell section cracked from end to end along the south side. Test discontinued.

In the tests of cast-iron rings No. 992A and 993A under a distributed load, the first crack appeared at the bottom at the maximum load and extended from end to end. The diagram is given in Fig. 20.

The effect of the lateral restraint is illustrated in the results of a preliminary test on Pipe No. 990. In this test, the hydraulic jack machine was not used, but I-beams were used to transfer the load from the 600 000-lb. testing machine to the saddle, since the box was too large to be placed on the bed of the machine. The distance of the saddle from the testing machine was such that only one-fifth of the load on the

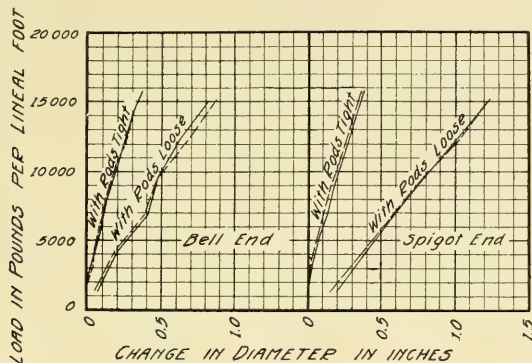


FIG. 21. LOAD-DEFLECTION DIAGRAMS FOR PIPE NO. 990 TESTED WITH AND WITHOUT LATERAL RESTRAINT.

machine was applied to the saddle. The load was run up to 16 200 lb. per lin. ft. on the pipe. At this load the change in the vertical diameter was 0.37 in. The load having been taken off, the rods holding the sides of the box were loosened and kept loose and pressure again applied. This time, at a load of 15 500 lb. per lin. ft., the average change in the vertical diameter was 1.05 in. with no sign of failure. When the load was removed there was a set of 0.65 in. at the spigot end. Four hours later this set was reduced to 0.30 in. The deflections in the two tests are shown in Fig. 21. This pipe was finally tested to destruction in the hydraulic jack machine, the first crack appearing at a load of 26 400 lb. per lin. ft. and the pipe finally carrying 37 500 lb. per lin. ft. as has already been described.

20. *Comparison and Discussion.*—Table 10 gives data of the distributed load tests of the cast-iron pipe and rings. The loads are given in pounds per foot of length of ring or pipe. The first crack for which the load is given appeared at the bell end in four pipes and at the spigot end in three. In No. 993 the crack ran from end to end and in No. 994, in which the pipe was cut and the spigot end placed in the bell

at the middle of the test box the first crack was at the bell. The breaking load was the maximum load and generally the indicated load of the testing apparatus fell off at once though in some cases the maximum load held until the deflections had been increased somewhat. M' is

TABLE 10.

CAST-IRON PIPE AND RING. DISTRIBUTED LOAD.

No.	W' Load at First Crack lb. per lin. ft.	W Breaking Load lb. per lin. ft.	M' $\frac{Wd}{16}$	M_0 $\frac{fbt^2}{6}$	Ratio $\frac{M_0}{M'}$
Pipes					
990	26 700	37 500	115 000	85 600	0.74
991	22 600	26 400	81 000	102 000	1.26
992	16 300	22 600	69 000	92 300	1.34
993	54 600	54 600	169 000	102 000	0.60
994	47 300	49 300	152 000	134 000	0.88
995	19 300	22 200	52 000	57 800	1.11
996	37 300	37 300	87 000	110 000	1.26
997	25 200	27 300	63 000	38 200	0.61
998	29 300	37 200	86 000	93 400	1.08
Rings					
992A	22 150	22 150	68 000	92 500	1.36
993A	29 250	29 250	90 000	104 000	1.16

the bending moment based on a uniform distribution of the breaking load over the horizontal section of the pipe and without any allowance for lateral pressure or restraint, as calculated by equation (7). M_0 is the resisting moment of a rectangular section of the pipe, calculated with the value of the modulus of rupture f determined from the concentrated-load tests of the rings cut from the pipe, except that for No. 990, from which no ring was cut, the average modulus of rupture determined from other pipe was used. In the expression for the resisting moment b is the length of pipe or ring and t is the average thickness. The last column gives the ratio of the resisting moment to the bending moment thus calculated. If M_0 properly measures the resisting moment and if the load is uniformly distributed over the horizontal section in both the longitudinal and transverse direction this ratio should be unity. If there is lateral pressure of the sand similarly distributed over the vertical section, the value of the ratio should be less than unity and would represent the $1 - q$ of equation (9). That is to say, if the ratio is 0.75, the lateral pressure would, by this method of reasoning, be 25%

of the vertical load. This treatment does not consider the effect of the greater strength and stiffness of the bell nor the effect of the stiffness of the bell upon the stresses in the barrel next to the bell. It is seen that there is a considerable variation in the value of this ratio. The higher values may be due to an uneven distribution of the load either longitudinally or transversely. In cases where the break occurred at one end or the other the lack of distribution seems evident. It is possible that in these cases the pipes were not so well bedded or that the sand was not so well packed about them. The lower values of the ratio indicate considerable lateral pressure. Evidently there is a great variation in condition of loading and the distribution of the pressures and it is possible that uneven resisting stresses of the pipe may account partly for this variation. This question will be discussed further on.

V. EXPERIMENTAL DATA WITH CONCRETE AND REINFORCED CONCRETE PIPE AND RINGS.

21. *Data of Tests.*—Table 11 gives dimensions and other data of concrete and reinforced concrete rings and Table 12 of reinforced concrete culvert pipes. Table 13 gives manner of loading of the plain and reinforced concrete rings and pipes and the loads carried. Tables 14 and 15 give the results of the concentrated load tests and Table 16 and 17 the results of the distributed load tests. These will be discussed for the two methods of loading under different heads.

In Fig. 22 to 27 (p. 47 to 55) are given diagrams of the amount of the vertical and horizontal deflections of the pipe and rings which were produced by the load per lin. ft. of length of pipe indicated by the vertical scale. These diagrams will be discussed for the two methods of loading.

22. *Concentrated-Load Tests.*—The method used in testing the cast-iron rings and pipes was used for the tests on concrete and reinforced concrete rings. As the plain concrete rings broke before there was an appreciable deflection no measurement of deflections was made. The modulus of rupture calculated from equation (10) is given in Table 14. The average modulus of rupture obtained from the 1907 tests of the small concrete beams was 195 lb. per sq. in. It is seen that the modulus of rupture from the ring tests is 20% higher than the modulus of rupture found in the beam tests. This variation is in the opposite direction from that found in the computation of the modulus of rupture of the cast-iron rings and the test beams. The variation in the values for the concrete is probably explainable by differences in the conditions

TABLE 11.
CONCRETE AND REINFORCED CONCRETE RINGS.

No.	Internal Diameter inches	Thickness inches	Length inches	Reinforcement		Age at Test days
				Kind	Per cent	
901	48	6	24	None	0	37
902	"	"	"	"	"	36
907	"	"	"	"	"	40
908	"	"	"	"	"	35
911	"	"	"	"	"	42
912	"	"	"	"	"	41
903	"	"	"	"	"	36
906	"	"	"	"	"	34
923	"	4	"	8 $\frac{1}{4}$ -in. corr. bar.	0.80	36
921	"	"	"	"	"	41
922	"	"	"	"	"	38
927	"	"	"	"	"	36
951	"	"	"	"	"	38
926	"	"	"	"	0.73	32
928	"	"	"	"	0.80	44
931	"	"	"	"	0.73	31
932	"	"	"	"	0.66	37
933	"	"	"	"	1.00	34
*934	"	"	"	"	0.80	98
952	"	"	"	"	1.00	36
953	"	"	"	"	0.89	45
*971	"	"	"	"	0.73	93
*972	"	"	"	"	"	87
†976	"	"	"	"	0.66	92
†977	"	"	"	"	"	90

* In these rings the reinforcing steel was flattened at top and bottom.

† In these rings stirrups were used, three at top and bottom.

TABLE 12.
REINFORCED CONCRETE CULVERT PIPE.

No.	Internal Diameter inches	Thickness inches	Length inches	Reinforcement		Age at Test days
				Kind	Per cent	
981	48	4	102	$\frac{1}{4}$ -in. corrugated bars	0.66	149
982	48	4	104	$\frac{1}{2}$ -in. corrugated bars	1.39	
983	48	4	102	$\frac{1}{4}$ -in. corrugated bars	0.66	
988	48	4	104	Clinton mesh, No. 3 wire	0.88	183
987	36	4	102	Fence wire	0.28	92

TABLE 13.
CONCRETE AND REINFORCED CONCRETE RINGS AND PIPE
DATA OF TESTS.

No.	Age at Test days	Manner of Loading	Critical Load lb.	Maximum Load lb.
Rings				
901	37	Concentrated		5 000
902	36	"		3 900
907	40	"		5 600
908	35	"		5 200
911	42	"		3 000
912	41	"		4 200
903	36	Distributed		61 000
906	34	"		88 000
926	32	Concentrated		5 700
928	44	"		7 100
931	31	"		5 000
932	37	"		6 000
933	34	"		6 350
934	98	"		6 300
952	36	"		4 700
953	45	"		7 200
971	93	"		8 250
923	36	Distributed	14 000	21 000
921	41	"	20 000	47 000
922	38	"	20 000	37 000
927	36	"	16 000	52 000
951	38	"	18 000	50 000
972	88	"	16 000	35 000
976	92	"	18 000	38 000
977	90	"	20 000	32 000
Pipe				
985 & 6		Distributed		200 000
981		"	166 000	268 000
982	149	"	129 000	215 000
983	118	"	106 000	202 000
988	183	"	76 500	262 000
987	92	"	76 500	202 000

attending setting in the two kinds of test pieces and by differences in the development of the stresses. As before, the value of the modulus of rupture obtained in the rings will be considered the better as a basis of comparison.

All failures occurred in either the top or bottom section of the cylinder as is to be expected from the relative value of the bending mo-

ment at these points and that at the extremities of the horizontal diameter. The breaks were sudden and the load which could be carried

TABLE 14.

CONCRETE RINGS. CONCENTRATED LOAD.

No.	Inside Diameter inches	Thickness inches	Load lb. per lin. ft.	Modulus of Rupture lb. per sq. in.
W. A. Slater's 1906 Tests.				
1	24	3	2530	304
2	24	5	7435	345
3	48	6	4215	254
4	48	10	12085	280
Av.				296
1907 Tests.				
901	48	6	2500	300
902	48	6	1950	234
907	48	6	1900	228
908	48	6	2300	276
911	48	6	1500	180
912	48	6	2100	252
Av.				245

after this break was very much less than the load at the time of the formation of the cracks.

The reinforced rings deflected considerably before final failure, as shown in Fig. 22. At a load of 1000 to 2150 lb. per lin. ft., fine cracks appeared in the tension face, generally at the top or bottom much as they appear in tests of reinforced concrete beams. With the application of higher loads, numerous fine cracks appeared on the tension faces at the top, bottom, and sides. Two forms of critical failure were apparent; one a tension failure of the reinforcing bars at the top or bottom of the ring (stretching beyond the yield point of the metal), and the other a failure of the concrete by the stripping and shearing of the concrete from the tension face. The latter form of failure may correspond to the diagonal tension failure in beams. Sometimes the concrete was split off along the reinforcing bars and these bars were straightened

from their original circular form. The following are notes of the tests:

No. 932. Fine cracks at top, bottom, and sides at 1500 lb. per lin. ft. At the maximum load of 3000 lb. per lin. ft. the deflection was 0.85 in. Later the concrete began to strip off.

No. 933. A fine crack appeared at the top at 1150 lb. per lin. ft., and at bottom at 1350 lb. per lin. ft., cracks at sides at 2100 lb per lin. ft. At the maximum load, 3170 lb. per lin. ft., the deflection was 0.8 in.

No. 934. A fine crack appeared at the top at 1300 lb. per lin. ft., at the bottom at 1550 lb. per lin. ft., and at the sides at 2050 lb. per lin. ft. At 3050 lb., per lin. ft. the deflection was 0.73 in. and at the maximum load, 3150 lb. per lin. ft., it was 1.05 in., and the cracks at the top and bottom soon opened up $\frac{1}{4}$ in., showing tension failure.

No. 928. A fine crack appeared at the bottom at 1400 lb. per lin. ft. and one at the top at 1600 lb. per lin. ft. These were soon followed by various fine cracks at the sides, bottom, and top. Deflection at 3450 lb. per lin. ft. was 0.88 in. and at the maximum load of 3550 lb. per lin. ft. $1\frac{1}{4}$ in.

No. 931. Fine cracks appeared at bottom and top at a load of 2200 lb. per lin. ft. At the maximum load, 2500 lb. per lin. ft. the deflection was 0.49 in. On further deflection the concrete inside the reinforcing bars at the top and bottom stripped off.

No. 952. A fine crack appeared at the bottom at 1000 lb. per lin. ft., at the sides at 1250 lb. per lin. ft., and at the top at 1450 lb. per lin. ft. At a load of 2250 lb. per lin. ft. the deflection was 0.97 in. At the maximum load, 2350 lb. per lin. ft., the concrete split off and the rods broke through at the top.

No. 953. A fine crack appeared at the bottom at a load of 1200 lb. per lin. ft., one at the top at 1350 lb. per lin. ft., and at the sides at 2200 lb. per lin. ft. Many fine cracks then appeared at bottom, sides, and top. At a load of 3300 lb. per lin. ft. the deflection was 0.77 in., and at the maximum load, 3600 lb. per lin. ft., 1.57 in.

No. 971. Fine cracks appeared at the top and bottom at 1500 lb. per lin. ft. and at the sides at 1750 lb. per lin. ft. At the maximum load, 4120 lb.

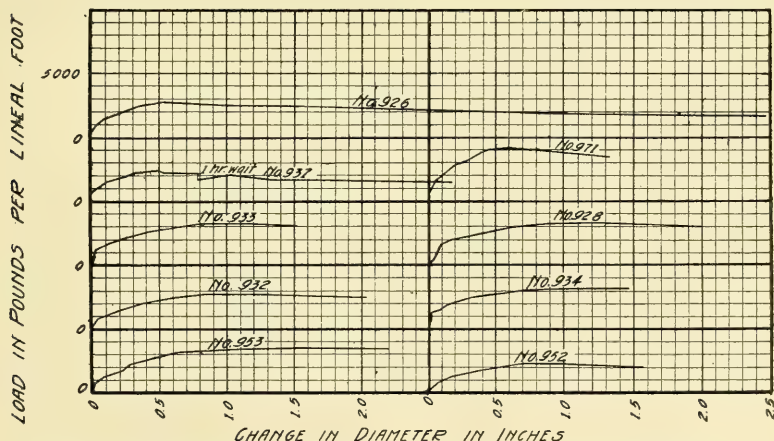


FIG. 22. LOAD-DEFLECTION DIAGRAMS OF REINFORCED CONCRETE RINGS. CONCENTRATED LOAD.

per lin. ft., the deflection was 0.6 in. Failed by tension in the steel at both top and bottom.

No. 926. Fine cracks appeared on the inner face at the top and bottom at a load of 1500 lb. per lin. ft. More fine cracks appeared near these at 2000 lb. per lin. ft., and the number of such cracks increased as the load was increased. Two cracks at the bottom increased in size, and one of these became quite large before the maximum load was reached, indicating a failure of the ring by tension in the steel. The maximum load was 2850 lb. per lin. ft. with a deflection of 0.53 in. The load then fell off, and as the deflections were increased by running the head of the testing machine down, part of the bottom sheared off on either side of the support and the concrete split off between the reinforcing rods and the inside surface of the ring.

Table 15 gives the calculated moments at the maximum loads. In part of them failure came by splitting of the concrete along the reinforcing bars. In some cases this was caused by imperfect fabrication, the bars being left too close to the interior face of the ring. The deflection curves have much the same characteristics. The maximum load gave a deflection of 0.5 to 0.8 in.

23. *Distributed-Load Tests of Concrete Rings.*—The plain concrete rings which were tested under a distributed load, bedded and covered with sand and restrained at the sides as already described, cracked at the bottom, top, and sides at an early load. With the continued application of the testing apparatus the load dropped off at first until at a somewhat greater deflection sufficient side restraint had been developed when the load again increased and continued to rise until

TABLE 15.

REINFORCED CONCRETE RINGS. CONCENTRATED LOAD.

No.	Load at First Crack lb. per lin. ft.	Maximum Load lb. per lin. ft.	<i>t</i> inches	0.16 <i>Wd</i>	0.87 <i>Aft</i>	Ratio
926	1500	2850	2.75	23 700	26 600	1.12
928	1400	3550	2.5	29 500	24 200	0.82
931	2150	2500	2.75	20 800	26 600	1.28
932	1500	3000	3.0	25 000	29 100	1.16
933	1170	3170	2.0	26 400	19 400	0.73
934	1300	3150	2.5	26 400	24 200	0.92
952	1000	2350	2.0	19 600	19 400	0.99
953	1200	3600	2.25	29 900	21 800	0.73
971	1500	4120	2.75	34 500	26 600	0.77

a considerable change had been made in the vertical and horizontal diameters when the test was discontinued. The load-deflection diagrams are given in Fig. 25 for No. 903 and No. 906. The following are notes of the tests of the two plain concrete rings.

No. 903. At a load of 3550 lb. per lin. ft. and a deflection of 0.09 in., a crack appeared at the bottom and the load dropped off to 2150 lb. per lin. ft. The indications were that the pressure of the sand of the bed was not well distributed. As the loading head was run down, cracks appeared at the east side of the ring at a load of 2150 lb. per lin. ft., the crack at the bottom opened to 1-16 in., and a crack formed at the top of the ring. At 2500 lb. per lin. ft. the crack at the bottom had opened to $\frac{1}{4}$ in. As the deflections were increased, the indicated load increased, and the cracks at the top, bottom, and sides increased in width, though the amount of the opening of the cracks at the sides could not well be measured. At a load of 8000 lb. per lin. ft. and a deflection of 1.45 in., the crack at the bottom was $\frac{1}{2}$ in. wide and that at the top $\frac{1}{4}$ in. At 13 000 lb. per lin. ft. another crack appeared at the west side 6 in. above the horizontal diameter, and crushing of the concrete at the extremities of the horizontal diameter became apparent. The crack at the bottom was $\frac{3}{4}$ in. wide and that at the top $\frac{1}{2}$ in. At 23 000 lb. per lin. ft. a crack appeared on the west side somewhat above the horizontal diameter. At 30 500 lb. per lin. ft. the vertical diameter was 3 ft. 7 $\frac{1}{2}$ in. and the horizontal 4 ft. 4 $\frac{1}{2}$ in. The test was discontinued, although the ring was still taking an increasing load.

No. 906. At a load of 4800 lb. per lin. ft. and a deflection of 0.03 in. a crack appeared at the bottom. At 4950 lb. per lin. ft. one appeared at the top. The load fell off to 4250 lb. per lin. ft. At a deflection of 0.3 in. and a load of 5050 lb. per lin. ft. cracks formed at the sides. These cracks gradually widened as the deflections and loads were increased. A new crack appeared at one side 8 in. above the extremity of the horizontal diameter at 15 500 lb. per lin. ft. and one in a similar place on the other side at 20 500 lb. per lin. ft. The crack at the bottom was $\frac{1}{4}$ in. wide at a load of 10 500 lb. per lin. ft., $\frac{1}{2}$ in. at 30 500 lb. per lin. ft., and $\frac{3}{4}$ in. at 40 500 lb. per lin. ft., and that at the top was 3-16, 5-8, and 1 in., at the same loads. At the maximum load of 44 000 lb. per lin. ft., there was crushing at the sides, and a circumferential crack formed. The vertical diameter was 3 ft. 8 $\frac{1}{2}$ in. and the horizontal 4 ft. 3 $\frac{1}{2}$ in.

It is evident that in these tests after the cracks appeared the rings did not act as a single structure but instead formed four pieces under equilibrium somewhat as shown in Fig. 23. Without adequate lateral restraint, equilibrium would not be maintained, and if the elasticity of

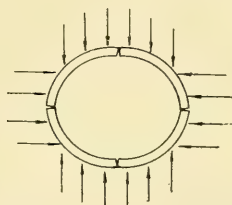


FIG. 23. ACTION OF UNREINFORCED PIPE UNDER DISTRIBUTED LOAD.

an embankment failed, stability would be lost. With the lateral pressure maintained, failure occurs by crushing of the concrete.

Fig. 24 gives the load-deflection diagram for the Jackson pipe. It will be seen that the curves are similar to those for the plain concrete rings.

24. *Distributed-Load Tests of Reinforced Concrete Rings.*—The tests on the reinforced concrete rings and pipe followed the method used in the tests of the cast-iron rings and pipe. The reinforced con-

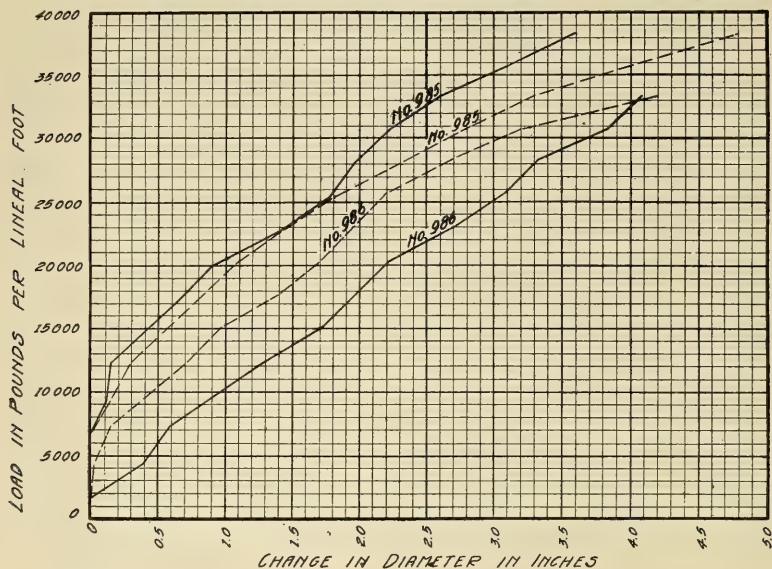


FIG. 24. LOAD-DEFLECTION DIAGRAMS FOR "JACKSON" PIPE.

crete rings were tested in the small box, using the 600 000-lb. testing machine, and the reinforced concrete pipes were tested in the hydraulic jack machine.

In the reinforced concrete rings cracks appeared early in the test on the tension side at the top and bottom, but the load continued to increase, and from this point to a load of 12 000 lb. per lin. ft. the reinforced concrete rings were much stiffer than the plain concrete rings and were evidently acting as reinforced concrete structures. During this stage numerous cracks appeared at the top, bottom, and sides similar to the tension cracks found in the test of reinforced concrete beams, and these increased in size. In part of the rings the cracks at the top and bottom increased in width to such an extent as to show failure by the steel becoming stressed past its yield point. In a number of the rings this did not occur, but instead the concrete was stripped or split off over the bars by the bars straightening out and pulling away from the interior concrete. In a few cases this was due to poor fabri-

cation, the bars being placed too close to the inner surface, but evidently this feature is a source of weakness in such construction. In No. 972, in which the bars at the top and bottom are made with a much flatter arc, and in No. 976 and 977, in which stirrups were used,

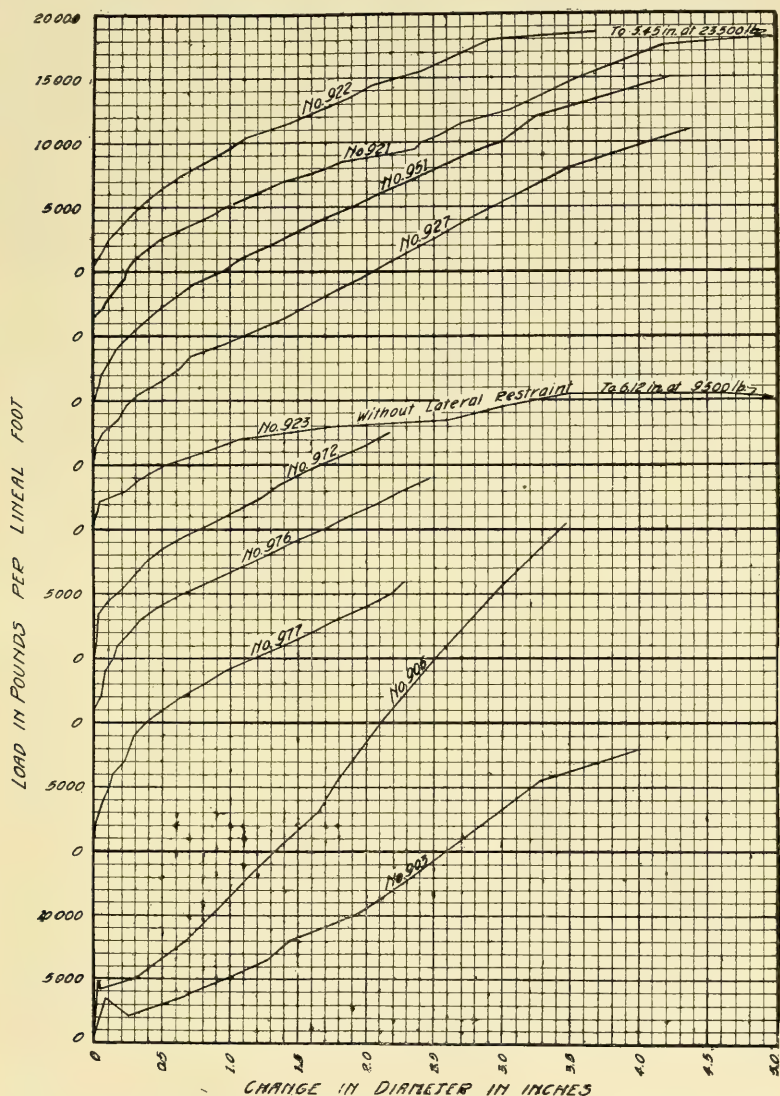


FIG. 25. LOAD-DEFLECTION DIAGRAMS FOR CONCRETE AND REINFORCED CONCRETE RINGS. DISTRIBUTED LOAD.

stripping of the concrete did not develop and the steel was stressed beyond its yield point. This was done also in No. 923. In all cases after the yielding of the steel or the stripping of the concrete the action of the ring in the test was much the same as in the tests of the plain concrete rings, and there seems to be no characteristic difference either in the loads or the amount of deflection except that the cracks did not open up wide at the sides and the angular displacement was distributed over a larger number of cracks. This view is borne out by the fact that

TABLE 16.

CONCRETE RINGS AND PIPE. DISTRIBUTED LOAD.

No.	Load at First Crack lb. per lin. ft.	Resisting Moment $\frac{1}{6} f b l^2$	$\frac{1}{16} W d$	Ratio	Maximum Load lb. per lin. ft.	Remarks
903	3 550	17 640	12 000	0.68	30 500	Load still going up. Test discontinued.
906	4 800	17 640	16 200	0.92	44 000	
985 & 986	2 000	12 250	6 630	0.54	40 000	
						Pipes from Jackson, Michigan.

* Assume $f = 245$.

for this stage of the test the deflections for all the reinforced concrete rings (see Fig. 25), fall between the lines for No. 903 and 906, the two plain concrete rings. It is evident that the action of the ring in the later part of the test is much the same as that of the plain concrete rings in that equilibrium is maintained by the lateral restraint of the sand and that the integrity of the structure is based upon the development and the maintenance of this lateral pressure. The following are notes of the tests.

No. 921. A fine crack appeared at the bottom at 3500 lb. per lin. ft. at a deflection of 0.14 in., two more at 4500, and one at top at 5000 lb. per lin. ft. The concrete began breaking off at the bottom at 5500 lb. and at the top at 7000 lb. per lin. ft., The rods broke through and the concrete stripped off at 9000 lb. per lin. ft. Circumferential cracks appeared at a load of 13 000 lb. per lin. ft. and others appeared at higher loads. Crushing at sides noticed at 16 500 lb. per lin. ft. and was crushing freely at a load of 23 500 lb. per lin. ft., when the deflection was 5.5 in.

No. 922. Fine crack appeared at bottom at 3500 lb. per lin. ft. and one at top at the same load. Other cracks appeared at top and bottom from 4500 to 6500 lb. per lin. ft. and at sides 7500 lb. per lin. ft. Rods broke through concrete at 12 500 lb. Load continued constant at 18 500 lb. per lin. ft.

No. 923. In this test the rods holding the sides of the box were loosened at times, leaving little side restraint. The load then would not increase until there was sufficient increase in the diameter to bring added pressure against the ring. A fine crack appeared at the bottom at 2250 lb. per lin. ft. and one at the top at 3000 lb. per lin. ft. Crushing at sides at 8000 lb. per lin. ft., and circumferential cracks formed. At 10 500 lb. per lin. ft. one rod broke out at bottom. Test discontinued. The large deflection due to this method of testing is shown in Fig 23.

No. 927. Fine crack at top and bottom at 3500 lb. per lin. ft. Other fine cracks formed at higher loads. Concrete stripped at bottom at 8500 lb. per lin. ft. Crushing at sides at 13 000 lb. per lin. ft. Freely crushing at sides at 16 500 lb. per lin. ft. Circumferential cracks formed at 23 000 lb. per lin. ft. Test discontinued at 26 000 lb. per lin. ft. and deflection of 4.4 in.

No. 951. Crack at bottom at 2950 and one at top at 4000 lb. per lin. ft. Other cracks soon formed. Stripping at bottom at 12 000 lb. per lin. ft. Crushing at sides at 14 000 lb. per lin. ft. Circumferential crack at top at 18 000 lb. per lin. ft. Test discontinued at 20 000 lb. per lin. ft. and deflection of 4.1 in.

No. 972. Fine crack at top at 4500 and three at bottom at 5500 lb. per lin. ft. Cracks opening up at 7500 lb. per lin. ft. Crushing began at 15 500 lb. per lin. ft. Test discontinued at 17 500 lb. per lin. ft.

No. 976. Fine crack appeared at top and bottom at 4000 and 6000 and 7000 lb. per lin. ft. Three tension cracks finally opened up wide at the bottom and two at the top. No evidence of bars pulling away from the ring. Crushing began at 19 000 lb. per lin. ft.

No. 977. Fine cracks appeared at top at 4000 and 6000 and at the bottom at 7000 lb. per lin. ft. More at 8000 lb. Bottom cracks were gradually opening at 9000 lb. per lin. ft. Failure by tension in steel at top and bottom was apparent at 16 000 lb. per lin. ft. No signs of bars pulling away. Crushing of concrete at 20 000 lb. per lin. ft. Test discontinued at 21 000 lb. per lin. ft.

25. *Distributed-Load Tests of Reinforced Concrete Pipe.*—The results of the distributed load tests of reinforced concrete pipe are quite similar to those of the tests of rings. Fine cracks became visible on the inner surface at the top and bottom at loads from 5000 to 11 000 lb. per lin. ft., the amount of the load depending upon the percentage of reinforcement, and numerous fine cracks soon formed. Circumferential cracks appeared at loads somewhat greater, generally next to the bell. At higher loads the cracks became somewhat larger though they did not open up until much later and in several cases in place of tension cracks opening diagonal shear cracks appeared, similar to those found in tests of reinforced concrete beams. As may be seen from Fig 26 and 27 there was evidently a change in the character of the load-deflection diagram at this time, and the critical load given in Table 17 is somewhat above the changes here noted. The amount of the deflection of the pipe at the time of the critical load averaged about 0.4 in. In several cases stripping of the concrete from the reinforcing bars at the top and bottom of the pipe occurred as in the tests of rings, but it generally appeared later and was not so serious as in the case of the

rings. The action of the pipe at loads beyond the critical load above referred to was quite different from that at previous loads, but the reinforcement evidently acted to hold the concrete together in the four

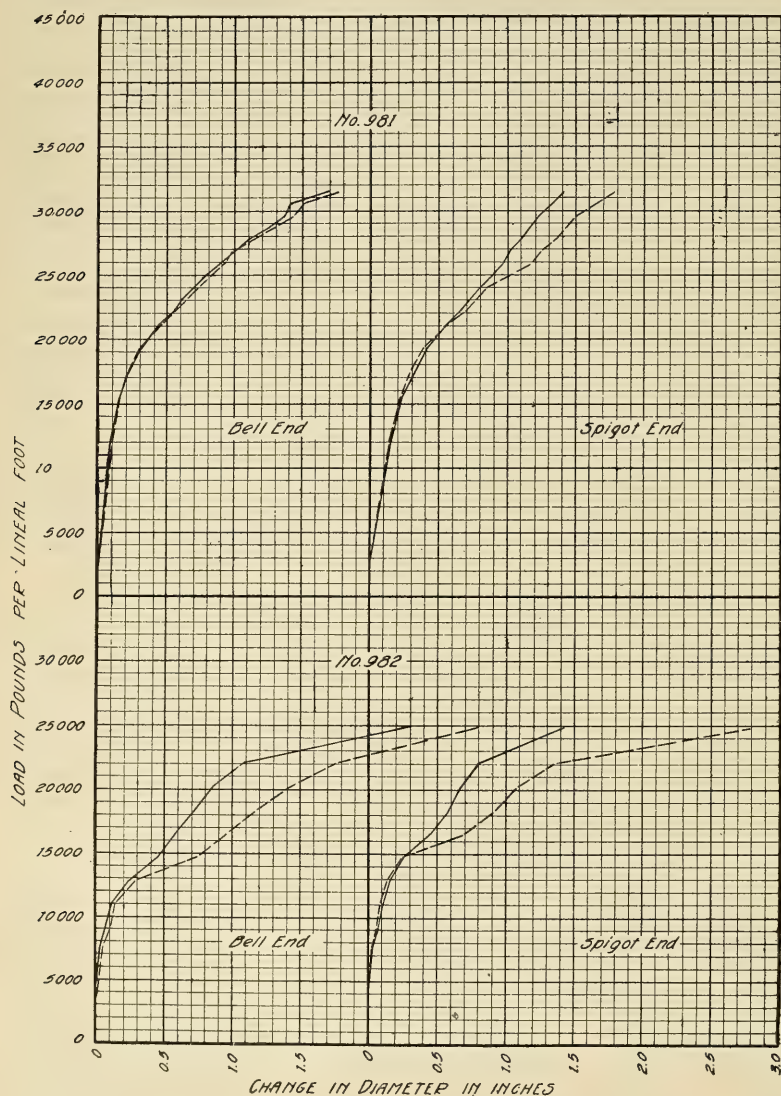


FIG. 26. LOAD-DEFLECTION DIAGRAMS FOR REINFORCED CONCRETE PIPES.
DISTRIBUTED LOAD.

quarters and the pressure was concentrated near the edge at the four quarter points, the pipe being held from total failure by the lateral restraint formed by the pressure of the sand at the sides of the pipe

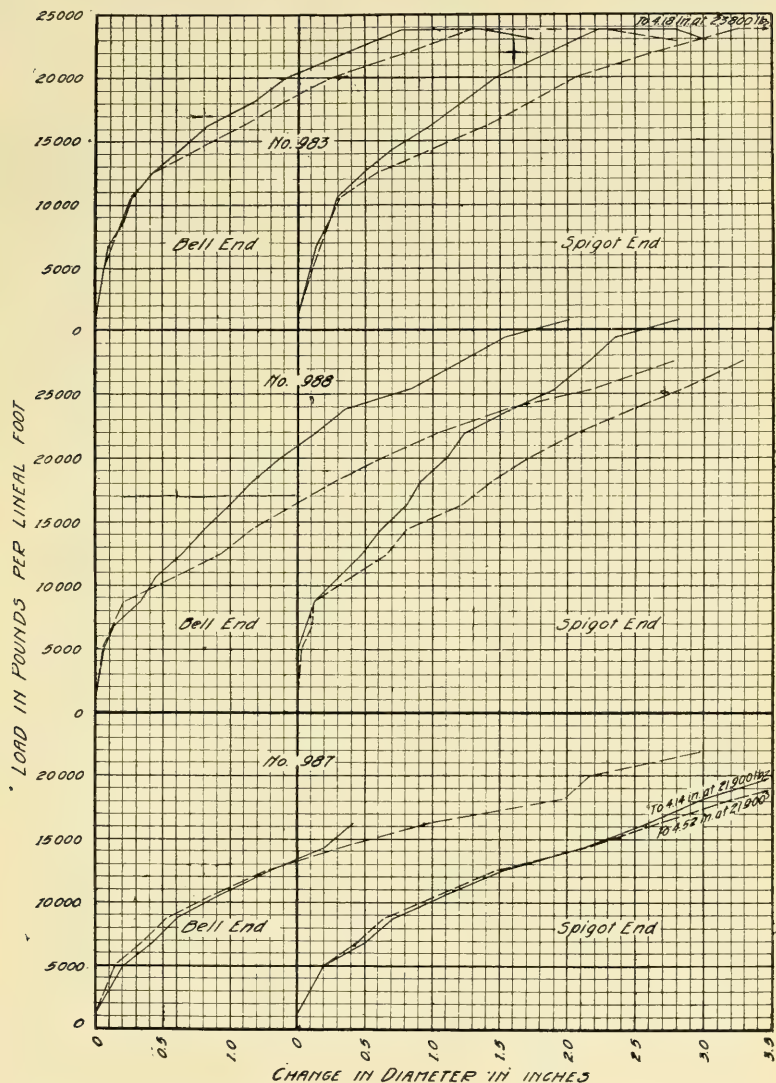


FIG. 27. LOAD-DEFLECTION DIAGRAMS FOR REINFORCED CONCRETE PIPES.
DISTRIBUTED LOAD.

in much the same way as has already been described for the reinforced concrete rings. Crushing of the concrete began to take place at the sides and top at loads near the maximum load, and after the vertical deflections had increased to three or four inches this crushing became marked, the load dropped off, and the test was discontinued. Fig. 28 and 29 give views of the condition of the reinforced concrete pipe at the end of the test. The following are notes of the tests.

Pipe No. 981. This pipe, which was reinforced with $\frac{1}{4}$ -in. corrugated bars, was given a preliminary loading in the testing box, the load being applied through I-beams used as levers. The center of the load was placed so that it was 11 in. closer to the bell than was the center of the pipe. In this test cracks first became visible at top and bottom at a load of 8100 lb. per lin. ft. These cracks extended from end to end of the pipe. Others appeared at top and bottom as the deflection increased, and circumferential cracks appeared in the bell at 11100 lb. per lin. ft. The loading was increased to 11600 lb. per lin. ft. at which point the I-beams commenced to buckle and the test was discontinued.

This pipe was afterward tested to destruction in the hydraulic jack testing machine. This time the load center was 8 in. off the pipe center toward the bell. The cracks above noted reopened and large radial and circumferential cracks appeared in the bell at a load of 18800 lb. per lin. ft. At 21500 lb. per lin. ft. one crack in the bottom was quite large from end to end and at 27000 lb. per lin. ft. crushing of the concrete commenced on the inside. Large pieces fell off the bell, exposing the steel when 29700 per lin.

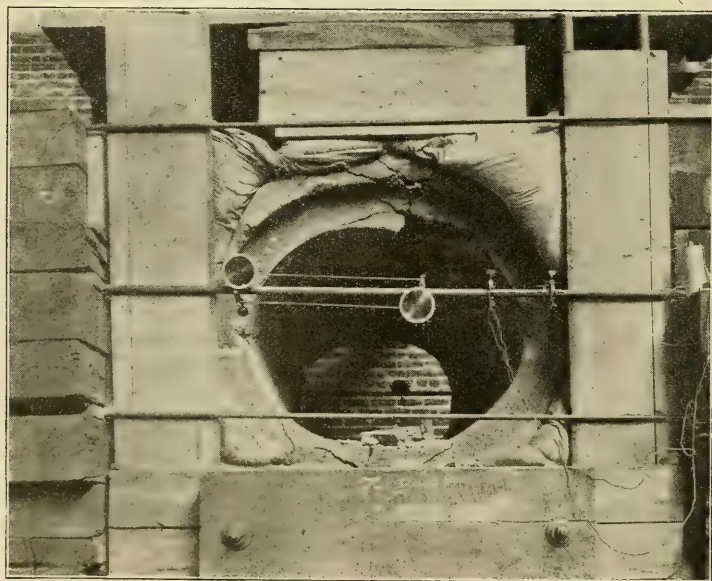


FIG. 28. VIEW OF REINFORCED CONCRETE PIPE AFTER TEST.

ft. was reached, and at 30 600 lb. per lin. ft. the maximum was reached and the test discontinued.

Pipe No. 982. This pipe was reinforced with $\frac{1}{2}$ -in. corrugated bars and was 149 days old at the time it was tested. This pipe was heated by steam coils to accelerate its hardening and an examination showed that the concrete was somewhat injured thereby. It was loaded 8 in. off center. At a load of 11 000 lb. per lin. ft. circumferential cracks appeared at top and bottom in the bell, and hair cracks appeared at top and bottom in the spigot end. The cracks opened up wider as the deflection increased and at 18 500 lb. per lin. ft.

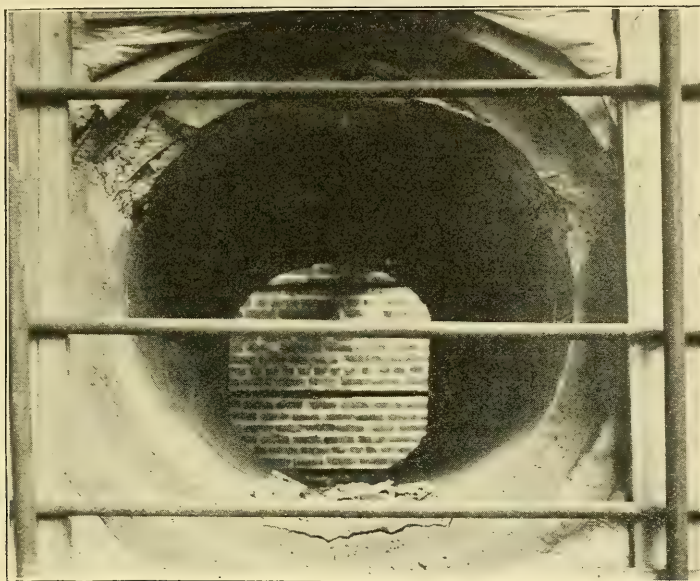


FIG. 29. VIEW OF REINFORCED CONCRETE PIPE AFTER TEST.

crushing commenced inside the bell end, soon extending from end to end of the pipe. At 22 200 lb. per lin. ft. diagonal cracks appeared at top and sides somewhat like shear cracks and soon crushing commenced on both sides about 20 degrees above the horizontal diameter. At a load of 25 000 lb. per lin. ft. the test was discontinued.

Pipe No. 983. This pipe was reinforced with $\frac{1}{4}$ -in. corrugated bars. The load center was 5 in. off the pipe center toward the bell. Tension cracks developed at the top in both ends at a load of 4900 lb. per lin. ft. and a crack appeared at the bottom at 6700 lb. per lin. ft. These cracks opened up wider and cracks appeared in the bell at a load of 10 400 lb. per lin. ft. Crushing commenced on both sides at 21 600 lb. per lin. ft. and at 23 400 lb. per lin. ft., the maximum, and the average change in the vertical diameter of 3.03 in. was noted, which increased rapidly as the load was held on the pipe.

Pipe No. 987. This test was made with the load $2\frac{1}{2}$ in. off center. This pipe, which was reinforced with fence wire, cracked end to end at both top and bottom at a load of 4950 lb. per lin. ft. At 6800 lb. per lin. ft. radial and circumferential cracks appeared all through the bell. As the deflection in-

creased the cracks opened wider and when a load of 10 500 lb. per lin. ft. was reached, crushing commenced on both sides near the bell and immediately extended end to end of the pipe. The bell commenced breaking away from the barrel at 20 000 lb. per lin. ft. and a maximum was reached at a load of 23 800 lb. per lin. ft. The appearance of this pipe in the later stages of the test indicated that the load was applied over a small portion of the surface, thus approaching a concentrated load. This may be the explanation of the large deflection found in this test.

Pipe No. 988. This pipe, which was loaded $1\frac{1}{2}$ in. off center toward the bell, was reinforced with Clinton wire mesh, the wire being No. 3. The first fine cracks appeared in both top and bottom and extended from end to end at a load of 6 700 lb. per lin. ft. At a load of 12 300 lb. per lin. ft. the first cracks had opened wider and shear cracks were appearing in the bottom of the spigot end. At 16 000 lb. per lin. ft. shear cracks opened in the top of the bell and tension cracks opened at both ends in the sides of the pipe. Crushing then started on the inside and a maximum was reached at a load of 30 900 lb. per lin. ft.

26. *Comparison and Discussion.*—Table 17 gives the loads at the appearance of the first crack, the maximum load carried, a so-called critical load, and a comparison of the critical load with the resisting moment of the pipe or ring acting as a reinforced concrete structure. By the first crack is meant the first crack which was observed, a very fine crack similar to the fine cracks which appear in reinforced concrete beams and which did not interfere with the strength or durability of the structure. The critical load is taken at the point where there is a marked change in the direction of the load-deflection diagram and where it becomes more nearly a straight line. There was at or about this load a noticeable change in the pipe, shown by the tension cracks enlarging or by the shear cracks forming. It seems evident that up to this point the resistance of the pipe is that of a reinforced concrete beam and that beyond this critical load the action of the reinforcement is principally to hold the parts of the ring together and the main resistance is the compressive strength of the concrete at the top, bottom, and side points against pressure induced by the arch action made possible by the lateral pressure and restraint of the sand at the sides. This view is corroborated by the amount of deflection and by the similarity of action and of deflection in the tests of the Jackson pipe and of the plain concrete rings. The column headed t gives the average distance from the center of the reinforcing bars to the compression face of the concrete at the top and bottom. No allowance has been made in the calculations for the greater size and stiffness of the bell. The column headed M' gives the bending moment calculated from the critical load on the basis that the load is evenly distributed over the horizontal section and that there is no side restraint. The column headed M_0 gives the resisting moment as a reinforced concrete beam by the form-

ula $0.87 Aft$ and is based upon the strength of the steel at its yield point as the controlling element. A is the area of reinforcement in a length of pipe of 1 foot, t is the distance from the center of the reinforcement to the compression face of the concrete and f is the strength of the steel at the yield point which is here taken to be 46 400 lb. per sq. in. for the rings, and 55 000 lb. per sq. in. for the pipe. For the condition that the reinforced concrete fails by the tension of the steel at its yield point, that the load is uniformly distributed over a horizontal section and that there is no lateral restraint, the ratio of the two moments should be unity. If the resisting moment due to the steel is not developed, as in the case of No. 982 where a large amount of reinforcement is used, this ratio will be greater than unity. In case there is side restraint the effect will be to reduce the ratio to an amount less than unity, provided both the horizontal pressure and the vertical pressure are uniformly distributed. For these conditions the amount of the ratio should be $1 - q$ given in equation (9). That is to say, if the

TABLE 17.
REINFORCED CONCRETE RINGS AND PIPE.
DISTRIBUTED LOAD.

No.	Load at First Crack lb. per lin. ft.	Critical Load lb. per lin. ft.	t inches	M' $\frac{1}{16} Wd$	M_0 $.87 Aft$	Ratio $\frac{M_0}{M'}$	Maximum Load lb. per lin. ft.
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Reinforced Concrete Rings.

*923	2 250	7 000	2.50	22 700	24 200	1.06	10 500
921	3 500	10 000	2.50	32 500	24 200	0.74	23 500
922	3 250	10 000	2.50	32 500	24 200	0.74	18 500
927	3 250	8 000	2.50	26 000	24 200	0.93	26 000
951	3 200	9 000	2.50	29 200	24 200	0.83	25 000
972	4 500	8 000	2.75	26 000	26 700	1.03	17 500
976	4 000	9 000	3.00	29 200	29 000	0.99	19 000
977	4 000	10 000	3.00	32 500	29 000	0.89	21 000

Reinforced Concrete Pipe.

981	8 360	19 500	3.00	63 500	34 700	0.55	31 500
982	10 960	15 000	3.00	48 800	72 600	1.49	24 800
983	4 950	12 500	3.00	40 600	34 700	0.86	23 800
988	6 700	9 000	3.00	29 200			31 400
987	4 950	9 000	3.00	29 200			23 800

*No lateral restraint.

ratio is 0.75 the lateral pressure would be 25% of the vertical pressure. In case the load is not uniformly distributed over the horizontal section this relation would not hold and the tendency of any concentration of the load near the center of the top of the pipe would be to give a smaller ratio than would otherwise be found. This is complicated by the probability of uneven distribution of the loads along the length of the pipe.

It is noticeable that No. 982 has a high ratio. This is the pipe with the large percentage of reinforcement and made of concrete which seems to have been injured in the curing. With this exception the ratios are generally less than unity and their average, 0.92, is not far from the average found in the distributed load tests of the cast-iron pipe and rings. Ring No. 923 in which the rods were kept loosened so that there was no lateral restraint has a somewhat higher ratio and agrees fairly well with the calculated value.

After the critical load has been passed the reinforced concrete ring will, under the conditions of the test, bear a considerably higher load than the critical load, though it must be understood that this load comes after the reinforcement ceases to act as in a reinforced concrete beam, that the cracks formed are such that the reinforcement may be exposed and the durability of the structure threatened, and that the strength of the pipe is dependent upon the maintenance of the lateral restraint of the sand at the sides.

VI. GENERAL COMPARISON OF RESULTS.

27. *Comparison of Methods of Loading.*—The tests under concentrated load for both the cast-iron rings and the reinforced concrete rings gave results which are consistent with analysis, both as to strength of the rings and as to the nature of the deflection curves and the amount of the deflection. The cast-iron rings broke suddenly, but the reinforced concrete rings (as shown in Fig. 22) maintained the maximum load until the deflections had increased materially. It is evident that the reinforced concrete structure may be deflected much beyond the amount which is produced by the critical load before final failure results.

In the discussion of the tests under distributed load for both the cast-iron pipe and the reinforced concrete pipe, it was noted that the determination of the resistance of the pipe to distributed load is much complicated by the uncertainty in the distribution of the load and in

the amount of lateral pressure which may be developed. It is worth while, however, to make a discussion of the observations and calculations to see what conclusions may be drawn. In Table 10 already referred to are given values of the bending moment and resisting moment developed in the cast-iron pipe, and in Table 16 values of these moments for the reinforced concrete pipe. As has already been noted, the expression used for the bending moment, $\frac{1}{8} Wd$, is based upon the assumption that the load is uniformly distributed over the horizontal section both longitudinally and transversely, and also that there is no lateral restraining pressure. The value of the resisting moment in Table 10 is based upon the modulus of rupture determined from the tests of the cast-iron rings under concentrated load. The value of the resisting moment of the reinforced concrete pipe in Table 16 is based upon the ordinary formula for strength of a reinforced concrete beam at the yield point of the reinforcement and does not consider that failure by diagonal shear or other cause may occur earlier.

Under the above assumptions the ratio of the resisting moment to the bending moment developed, as given in the above table, should be unity. If a lateral pressure acts, the ratio should be less than unity and its value would correspond to the $1 - q$ of equation (9). If the lateral pressure is 25% of the vertical pressure, both being assumed to be uniformly distributed, the ratio would be 0.75. If, however, the load is not uniformly distributed over the horizontal section the effect would be to give a larger ratio in the calculations made than would be found if the actual distribution of the load were known and used. The effect of the bell itself may possibly make the resisting moment of the pipe smaller than is assumed in the tables.

The average value of the ratio in the table for the cast-iron pipe is somewhat less than unity. It has been suggested that the higher values of this ratio may be due to uneven bedding and distribution of the load and this is borne out by some of the observations of the test. The lower values of the ratio indicate the presence of considerable lateral pressure, and the effect of no lateral pressure upon deflections is quite apparent in the test of cast-iron pipe No. 990, and in the reinforced concrete ring No. 923, where the horizontal restraining rods were kept loosened.

Evidently there is more or less variation in the conditions of the test and probably also in the resisting strength of the pipe. In the reinforced concrete rings and pipe the selection of the critical load given in Table 16 is dependent somewhat upon judgment, but the values

have been compared with the conditions in the concentrated load tests, and changes which may be made by different individuals would not affect the results materially. The value of the ratio of the two moments is seen to be quite similar to those given in the table for the cast-iron pipe, and its average is under unity. Evidently the conditions relating to the distribution of load and the effect of the lateral pressure are similar to those found in the test of cast-iron pipe. For high percentages of reinforcement or with steel of a high elastic limit, like drawn wire, the pipe is likely to fail by other forms of failure than through the steel being stressed beyond its yield point.

In the reinforced concrete pipe beyond the so-called critical load the action of the structure, as has already been stated, is quite different and the final failure is through crushing of the concrete. It would seem that this strength is available in an emergency, though the condition of the concrete in reference to cracks and defects may be such as to affect the durability of the structure.

28. *Measure of Strength of Pipe.*—From these tests it seems evident that the lateral restraint is considerable and that the pressure exerted at the sides aids considerably in holding the pipe from large deflections and thus strengthens it materially, but at the same time the apparent effect of this is largely counteracted by the lack of uniformity in the distribution of the load and the lateral pressure, which results in making the bending moment of the vertical load larger than the assumed moment. Any reduction in the ratio $1 - q$ below unity which may be found in the tests may be considered to be merely an added safeguard. It will probably be best then to use $\frac{1}{18} Wd$ for the bending moment coming upon such a pipe when the bedding and later filling are well done, considering any reduction in the ratio of moments here discussed only as a margin of safety. In case of careless or indifferent bedding or filling the lack of uniformity of distribution of pressure transversely and longitudinally will require that a higher bending moment than $\frac{1}{18} Wd$ be used.

The strength of cast-iron pipe may be calculated by using, in the expression for resisting moment, a value of the modulus of rupture, say, 25% less than the modulus of rupture obtained by breaking small beam test specimens of the same metal. The effect of the presence of the bell is somewhat uncertain but it is quite probable that greater strength could be obtained by distributing the metal of the bell throughout the barrel of the pipe. It seems probable, too, that as ordinarily laid in the embankment the stiffness of the bell will inter-

fere with the distribution of pressure over the bed and thus reduce the strength of the culvert. It should be noted, too, that it is probable that the quality of the cast-iron pipe tested was better than the ordinary run of cast-iron culvert pipe used by railroads. In the tests the cast-iron pipe failed at the maximum load and the load sustained dropped off suddenly, indicating that there would be a complete collapse under a dead load.

The critical strength of reinforced concrete culvert pipe where the reinforcement does not exceed, say 0.75 of 1% and is of medium steel may be measured by the resisting moment calculated by the ordinary beam formula. This, of course, is with good concrete. The resistance against diagonal tension and stripping of the concrete over the bars may be improved by flattening the arc around the top and bottom and possibly by the use of stirrups at these points. The actual load which the reinforced concrete pipe will take above this critical load is a great safeguard. The property which a reinforced concrete beam has of holding a load near its maximum load through a considerable deflection may be of great value in case the earth at the sides yields and the pipe must follow it to get the benefit of side restraint.

29. *Loads and Failures.*—The distributed load tests herein described were made with the filling of sand carefully placed and packed. It is evident that the condition of the bedding and filling and also the nature of the materials used in filling over the pipe will have a great influence upon the amount of the load and upon its distribution. The experience in the breaking of vitrified pipe sewers is analogous. Many cases of breakage of lines of pipe sewer have been reported in diameters from 18 in. upward. These instances have occurred in rock and clay more generally, though such failures are found in sand and quick-sand. The load which will come upon such sewer pipe from the trenches will vary with the manner of filling and the nature of the soil, as has already been suggested. Failures of cast-iron pipe under high embankments have been reported. In some of these cases the loose rock which was used for filling produced high loads. The effect of the manner of filling and of the nature of the material and the cause and prevention of such breakages would make a long paper by themselves. It is hoped, however, that the publication of this paper will bring to light instances of the failure of culvert pipe and sufficient data to throw light upon the loads which were produced by the embankment. If engineers will report the circumstances attending such failures, the height of the embankment, the conditions of the bedding, the nature of filling, the

nature of the materials placed in the filling, and the time which had elapsed after construction, the information will be very helpful in planning new structures, particularly where new types of construction like concrete and reinforced concrete are to be used. Such data will add much to the information given in this paper.

30. *Summary.*—From the tests and the discussions it would seem evident that among the facts brought out are the following :

1. The cast-iron rings broke under a concentrated load at a calculated modulus of rupture 25% less than the modulus obtained from rectangular test pieces cut from the rings. The average value of the modulus of rupture for the ring tests was 27 000 lb. per sq. in., and it should be noted that this value was obtained with an excellent quality of iron.

2. The cast-iron pipe loaded in sand broke suddenly at one end or the other, finally breaking through the entire length of the pipe. Upon failure the load dropped materially and there was little further strength to the structure. The stiffness of the bell affected the deflections and acted to prevent a uniform distribution of stress throughout the length of the barrel. The presence of the bell adds to the difficulties of securing a uniform distribution of the load, and detracts from the strength of the pipe.

3. The plain concrete rings broke under slight deflections at loads which agreed well with the calculated strength, both under concentrated load and distributed load. In the testing box under the restraining lateral pressure these rings held high loads after the segments had been deflected considerably from their original position, finally breaking by crushing of the concrete under conditions shown in Fig. 23.

4. The reinforced concrete rings in the concentrated load tests held their maximum loads or about their maximum loads through a considerable deflection, thus showing a quality which is of value when changes in earth conditions permit a gradual yielding of the surrounding earth. The calculated restraining moment agrees fairly well with the calculated bending moment.

5. The reinforced concrete rings and pipes tested under distributed load made a satisfactory showing. The so-called critical failure may occur by either tension failure in the steel or a diagonal tension failure (ordinarily called shearing failures) in the concrete. A flattened arc for the reinforcement where it approaches the inner face is of assistance and stirrups may be of some value. Beyond the critical load the re-

inforcement is of service in distributing the cracks and in holding the concrete together. Final failure is by crushing of the concrete in much the same way as was obtained with the plain concrete rings. The additional strength beyond the critical load may be taken into consideration in selecting the factor of safety or working strength.

6. The restraint of the sand in the tests is very important, and the effect is to reduce the bending moment developed by a given vertical load, or, as it would be commonly stated, to add strength to the pipe. The degree of permanency of this side restraint is uncertain. It seems evident in these tests that the distribution of the pressure, both horizontal and vertical, was not uniform, and that with the usual method of placing a pipe in an embankment, and especially when other materials than sand are used, the distribution would be even less uniform than here found. In view of this it will be well in making calculations and designs to use the formula $\frac{1}{16} Wd$ for the bending moment, thus considering that the side of restraint is offset by the uneven distribution of the load, any surplus from this being considered merely an additional margin of safety. For pipes poorly bedded and filled a larger bending moment than $\frac{1}{16} Wd$ should be used.

7. The method of bedding and laying pipes and the nature of the bed and the surrounding earth have a great effect upon the bending moment developed and upon the resistance of the pipe to failure. If the method of laying, or the hardness of the soil below, or the condition of the settlement of the pipe is such that the pipe is supported only or mainly along an element of the cylinder at the bottom the bending moment developed will be greatly increased over that of a uniformly distributed support. If the greatest supported pressure comes at points well to the side of this bottom element, as may be obtained by careful bedding, the bending moment is reduced. It is also plain that the bell should be left free from pressure at the bottom. It is possible that the presence of the bell detracts from the strength of the pipe. Any action in filling which increases the lateral restraint against the pipe will add to the security of the structure.

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- Bulletin No. 1.* Tests of Reinforced Concrete Beams, by Arthur N. Talbot. 1904. (*Out of print*).
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- Bulletin No. 23.* Voids, Settlement and Weight of Crushed Stone, by Ira O. Baker. 1908.

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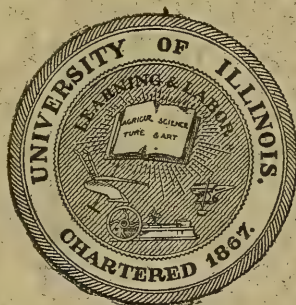
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BULLETIN NO. 23

VOIDS, SETTLEMENT AND WEIGHT OF CRUSHED STONE

BY

IRA O. BAKER



UNIVERSITY OF ILLINOIS
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BULLETIN No. 23

MAY 1908

VOIDS, SETTLEMENT AND WEIGHT OF
CRUSHED STONE

BY IRA O. BAKER, PROFESSOR OF CIVIL ENGINEERING

INTRODUCTION

Crushed stone has become an important material of construction in modern engineering work. The chief causes for this are the great increase in the use of plain and reinforced concrete, and the increased activity in macadam road construction. The advances made in these lines have been so rapid that crushed stone has suddenly changed from a minor material to one of first importance in modern engineering construction. This has been done in such a short time that the present knowledge of the properties of crushed stone is entirely inadequate; and the determination of its weight, voids, and settlement, and the variations of these have never been attempted on any adequate scale, so far as the writer has been able to ascertain. Before commencing this article a diligent search was made of engineering literature for information upon this subject. No definite information was found concerning the weight of a cubic yard of stone of different sizes (except one item as noted in Appendix I) or the amount of settlement in transit. The only other reference on the subject was the request of a correspondent in one of the leading engineering journals for information regarding the weight of crushed stone. In answer a wide range of limits was given with the explanation that as no definite values were known, the general practice was to assume some value within these limits.

The need for reliable data on these subjects is very apparent to the engineer who makes designs and estimates. Accustomed to use all other materials, both of engineering and everyday life, and to deal with standard units of weights and measures, he finds here that there are no standards at all. For instance, practically all of the quarries sell stone by the yard, but the so-called yard in one place is not always the same as the yard at some other place. In most cases a certain weight is taken as a yard; but these weights are generally arbitrary amounts that are supposed to approximate the true value, and they differ for different localities and for the different kinds and sizes of stone. Consequently, the number of yards and therefore the cost of the stone for the same piece of work would differ according to the location of the stone supply. Furthermore, this ambiguity may cause difficulties to arise between the producer, the carrier, and the consumer. The producer measures the volume loose in the car after it is loaded from the crusher. The railway then weighs the cars and computes the number of cubic yards by assuming the weight of a yard. The consumer receives the invoice from the producer, and the freight bill from the railway, and tries to check them, but generally finds they do not agree. So it is evident that this lack of standards entails possibilities of constant controversy between the shipper, the railroad, and the consumer.

Again, it is well known that the volume of crushed stone shrinks in transit; and to make accurate estimates the engineer should know the probable amount of this shrinkage. If a certain number of yards of tamped or consolidated stone are required for a pier or for a certain length of macadam road, it is necessary to know how many yards to order at the quarry so as to have the required amount in the structure. As done at present, the engineer to be on the safe side usually orders considerably more than he thinks is enough, and even then he sometimes finds he has not made allowance enough.

In order to establish a definite standard for the different sizes and varieties of crushed stone, tests should be made until sufficient data have been accumulated to determine a definite value for the weight of a cubic yard of crushed stone under various conditions, or to establish a coefficient by which either the weight of a

cubic foot or a cubic yard of the solid stone, or its specific gravity, can be multiplied to give the weight per cubic yard of crushed stone. It is obvious that to make the results of the greatest value will require a very large number of observations under a variety of conditions. It is the purpose of this article to give the results of a few tests along these lines.

Of the data hereinafter referred to, the observations on Chester stone and part of those on Joliet stone were made by Mr. Albert J. Schafmayer, a senior student in Civil Engineering, during the summer of 1906, while employed by the Illinois Highway Commission in connection with constructional work. A brief summary of Mr. Schafmayer's results was published in the report of A. N. Johnson, State Highway Engineer, in the first annual report of the Illinois Highway Commission. The observations on Kankakee stone and part of those on Joliet stone were made by Mr. Benjamin L. Bowling, an employee of the Engineering Experiment Station, during the fall of 1907. None of the investigations could have been made except for the generous cooperation of the officials of the State Penitentiaries at Chester and Joliet, and of the McLaughlin-Mateer Company of Kankakee.

Some observations were taken at Chicago and at Gary, Illinois, but unavoidable conditions at these plants prevented a completion of the work, and the results obtained are too incomplete to be of any considerable value, and hence are not further referred to.

THE STONE

The observations referred to in this article relate wholly to limestone, although in the appendix some data are given concerning trap. The limestones experimented with were those quarried at Chester, Joliet, and Kankakee.

The Chester stone is a rather coarsely granulated gray limestone of the lower carboniferous group, and is quarried in the grounds of the State Penitentiary at Chester, on the Mississippi River, about half way between St. Louis and Cairo.

The Joliet stone is a compact, fine-grained magnesian limestone of the Niagara series, and is quarried in the grounds of the State Penitentiary at Joliet, about 40 miles southwest of Chicago. The output of the crusher consists of 28 per cent 3-in. stone, 53 per cent 2-in., and 17 per cent $\frac{1}{2}$ -in.

The Kankakee stone is a coarse-grained argillaceous limestone of the Niagara group, and is quarried at Kankakee, on the Kankakee River, about 55 miles south of Chicago.

DIVISIONS OF THE SUBJECT

The subject will be considered under the following heads: I. Specific gravity; II. Absorptive power; III. Percentage of voids; IV. Settlement in transit; V. Weight per cubic yard; VI. Coefficients for determining the weight of crushed stone.

I. SPECIFIC GRAVITY

A knowledge of the specific gravity of a stone is useful in determining the per cent of voids in broken stone; and the easiest way to determine the weight of a cubic unit of solid stone is to find its specific gravity.

$$\text{Specific gravity} = \frac{W_a}{W_a - W_w}$$

in which W_a is the weight of a fragment weighed in air, W_w the weight of the same fragment suspended in water. If the stone is porous to any considerable extent, the weight in water should be determined so quickly that the absorption during the weighing will be inappreciable.

Samples of stone were collected from the various parts of the Joliet, the Kankakee, and the Chester quarries which were being worked to produce the broken stone considered in the later parts of this paper. The values of the specific gravity are given in Table 1.

II. ABSORPTIVE POWER

A knowledge of the amount of water absorbed by a stone is useful in determining the voids by the method of pouring in water, and is also useful in correcting the weight of wet stone.

The absorption was determined by thoroughly drying a specimen, weighing it, immersing it in water for 96 hours, drying with blotting paper, and weighing. The results are given in Table 2.

III. PERCENTAGE OF VOIDS

The per cent of voids in broken stone of different sizes has an important bearing upon the amount of cement and sand required

TABLE 1
SPECIFIC GRAVITY OF LIMESTONE

Ref. No.	Location of Quarry	Specific Gravity	Observer	Position in Quarry
1	Joliet	2.77	Schafmayer	Near Center, 4 to 6 ft. deep
2	"	2.78	"	" " " "
3	"	2.70	"	25 ft. W. of C., 4 ft. to 6 ft. deep
4	"	2.69	"	" " " "
5	"	2.72	"	" E. " "
6	"	2.74	"	" " " "
7	"	2.71	"	" N. " "
8	"	2.70	"	" " " "
9	"	2.63	"	" S. " "
	Mean	2.71		
10	Joliet	2.74	Bowling	25 ft. S. of C. of floor 30 ft. deep
11	"	2.68	"	" " " "
12	"	2.74	"	" At " "
13	"	2.73	"	" " " "
14	"	2.69	"	25 ft. N. " "
15	"	2.70	"	" " " "
	Mean	2.71		
16	Chester	2.67	Schafmayer	N. end over 25 ft. deep
17	"	2.58	"	50 ft. S. of the preceding, over 25 ft. deep
18	"	2.59	"	100 " " " "
19	"	2.49	"	Center near top
20	"	2.66	"	N. end " "
21	"	2.50	"	S. " " "
22	"	2.48	"	Center " "
	Mean	2.57		
23	Kankakee	2.62	Bowling	S. end at top
24	"	2.64	"	" " " "
25	"	2.65	"	" " 20 ft. deep
26	"	2.65	"	" " " "
27	"	2.56	"	" " 40 ft. "
28	"	2.56	"	" " " "
29	"	2.60	"	N. " at floor
30	"	2.62	"	" " " "
	Mean	2.61		

in making concrete; and the per cent of voids in connection with the weight of a unit of solid stone is useful in determining the weight of a unit of volume of broken stone.

The percentage of voids may be determined in either of two ways: (1) by pouring in water; and (2) by computation from the specific gravity and the weight of a volume of broken stone.

1. *By Pouring in Water.* Determine the weight of water a given vessel will contain, then fill the vessel with broken stone, and determine the weight of water that can be poured into the

TABLE 2
ABSORPTIVE POWER OF LIMESTONE

Ref. No.	Kind of Stone	Weight in lb. per cu. ft.	Absorption		Position in Quarry
			lb. per cu. ft.	Per cent by Weight	
1	Joliet	170.66	1.23	0.72	25 ft. S. of C. of the floor, 30 ft. deep
2	"	166.98	.79	0.47	" " " "
3	"	170.85	.86	0.50	At Center " "
4	"	170.41	1.25	0.73	" " " "
5	"	167.61	1.13	0.68	25 ft. N. " "
6	"	168.60	1.28	0.76	" " " "
	Mean	169.18	1.09	0.64	
7	Kankakee	163.49	1.87	1.14	S. end at top
8	"	164.92	1.98	1.20	" " " "
9	"	165.11	2.66	1.61	" " 20 ft. deep
10	"	165.30	2.81	1.70	" " " "
11	"	159.81	4.49	2.81	" " 40 ft. "
12	"	159.68	4.67	2.92	" " " "
13	"	162.24	2.99	1.84	N. " at floor
14	"	163.73	2.85	1.74	" " " "
	Mean	163.04	3.04	1.84	
15	Chester	167.0	.69	.31	
16	"	165.8	1.22	.74	
17	"	164.5	1.89	1.15	
	"	165.1	1.34	.81	
18	"	161.5	2.54	1.57	
19	"	161.5	2.34	1.45	
20	Mean	164.2	1.67	1.01	

interstices of the broken stone. The ratio of the first amount of water to the second is the proportion of voids.

In this method three sources of error require consideration. (a). In pouring in the water, part of the contained air is not driven out; and therefore the resulting per cent of voids is too small. The error from this source may be reduced, if not entirely eliminated, by pouring the stone into the water; but this procedure introduces a new error, since the stone will not pack to the same degree as in the ordinary method of filling a vessel or bin with broken stone, and hence the result of pouring the stone into the water will also give too large a per cent of voids. (b). If the stone absorbs water during the test the apparent per cent of voids will be too great. (c). If the vessel has a wide mouth, as almost necessarily it should have, there will be a likelihood of considerable error in telling when the vessel is exactly full of stone and also of water. The resulting error may make the per cent of voids either too large or too small.

2. *By Computation.* Determine the weight of a known volume of broken stone. Compute the weight of an equal volume of the solid stone by multiplying the known volume by the weight of an equal volume of water and by the specific gravity of the stone. The difference between the weight of the volume of solid stone and that of the broken stone is the weight of stone equal to the volume of the voids. The ratio of this weight to the weight of the given volume of broken stone is the proportion of voids.

This method is subject to the error of determining when the vessel is exactly full of stone. In practice it is more complicated than the preceding method, but it is more exact.

Table 3 gives the per cent of voids for three sizes of Chester limestone determined by the two methods referred to above, by two independent observers for different methods of filling the vessels with broken stone; and Tables 4 and 5 the same for Joliet and Kankakee limestone, respectively. In each case the results are corrected for the absorption of the stone. Precautions were taken also to eliminate absorption by the walls of the vessel used. The distance of drop employed in filling the vessel corresponded to that employed at the time in loading cars of broken stone.

TABLE 3

PERCENTAGE OF VOIDS OF CHESTER LIMESTONE

No. of Test	Size of Stone	Method of Filling	Wt. of Stone lb.	Wt. of Stone and Water, lb.	Wt. of Water lb.	Vol. of Water cu. ft.	Per Cent of Voids	
							By Pouring in Water	From Specific Gravity
By Use of Vessel Containing 27 cu. ft.								
1	$\frac{3}{4}$ -in. Scr.	15 ft. drop	2430	3150	720	11.52	42.7	43.9
2	"	"	2395	3095	700	11.20	41.5	42.4
3	"	"	2435	3140	705	11.28	41.8	43.8
Mean							42.0	43.4
4	2 in.- $\frac{3}{4}$ -in.	15 ft. drop	2320	3110	790	12.64	46.8	46.4
5	"	"	2375	3165	790	12.64	46.8	45.8
Mean							46.8	46.1
6	3 in.-2 in.	15 ft. drop	2370	3160	790	12.64	46.8	45.3
7	"	"	2390	3185	795	12.72	47.2	44.8
Mean							47.0	45.0
By Use of Vessel Containing 2.6 cu. ft.								
8	$\frac{3}{8}$ -in. Scr.	Shoveled	226.5	293	66.5	1.06	41.0	45.8
9	"	"	227.0	293	66.0	1.06	40.6	45.7
10	"	"	216.5	283	66.5	1.06	41.0	48.9
Mean							40.9	46.8
11	$\frac{3}{4}$ -in. Scr.	"	214.5	286	71.5	1.14	44.0	48.6
12	"	"	210.5	284	73.5	1.17	45.2	49.6
Mean							44.6	49.1
13	"	20 ft. drop	229.0	293	64.0	1.025	39.4	45.2
14	2 in.- $\frac{3}{4}$ -in.	Shoveled	204.0	286	82.0	1.31	50.5	51.2
15	"	20 ft. drop	237.0	306	69.0	1.10	42.5	43.3
16	3 in.-2 in.	Shoveled	212.0	291	79.0	1.265	48.7	49.3
17	"	20 ft. drop	245.0	313	68.0	1.09	41.6	41.3

TABLE 4
PERCENTAGE OF VOIDS OF JOLIET LIMESTONE

No. of Test	Size of Stone	Method of Filling	Wt. of Stone lb.	Wt. of Stone and Water, lb.	Wt. of Water lb.	Vol. of Water cu. ft.	Per Cent of Voids		
							By Pouring in Water	From Specific Gravity	
By Use of Vessel Containing 2.34 cu. ft.									
1	½-in. Scr.	8 ft. drop	208.75	270.75	62.00	0.99	42.3	47.6	
2	“	“	208.75	271.25	62.50	1.00	42.7	47.6	
3	“	“	207.75	270.25	62.50	1.00	42.7	47.9	
							Mean	42.6	47.7
4	2 in.-½-in.	8 ft. drop	218.75	285.75	67.00	1.07	45.8	45.1	
5	“	“	221.75	288.50	66.75	1.07	45.6	44.3	
6	“	“	218.50	285.75	67.25	1.08	45.9	45.1	
7	“	“	220.00	286.75	66.75	1.07	45.6	44.8	
							Mean	45.7	44.8
8	3 in.-2 in.	8 ft. drop	227.25	291.75	64.50	1.03	44.1	43.0	
9	“	“	219.25	287.75	68.50	1.10	46.8	45.0	
10	“	“	222.25	290.25	68.00	1.09	46.5	44.2	
11	“	“	212.00	282.25	70.25	1.12	48.0	46.8	
							Mean	46.3	44.7
12	½-in. Scr.	4 ft. drop	215.75	276.00	60.25	0.96	41.1	45.8	
13	“	“	218.75	279.25	60.50	0.96	41.3	45.1	
14	“	“	209.25	273.00	63.75	1.02	43.5	47.5	
15	“	“	208.25	272.25	64.00	1.02	43.7	47.7	
							Mean	42.4	46.5
16	2 in.-¾-in.	4 ft. drop	203.00	277.25	74.25	1.19	50.7	49.0	
17	“	“	209.75	283.25	73.50	1.15	50.2	47.4	
18	“	“	209.75	282.75	73.00	1.17	49.9	47.4	
19	“	“	212.25	283.75	71.50	1.14	48.8	46.7	
20	“	“	213.25	284.00	70.75	1.13	48.3	46.5	
							Mean	49.6	47.4
21	3 in.-2 in.	4 ft. drop	221.25	291.25	70.00	1.12	47.8	44.5	
By Use of Vessel Containing 2.43 cu. ft.									
22	3 in.-2 in.	4 ft. drop	211.25	287.75	76.50	1.22	50.4	49.0	
23	“	“	216.25	289.25	73.00	1.17	48.1	47.7	
24	“	“	212.75	285.75	73.00	1.17	48.1	48.6	
							Mean	48.6	47.5

TABLE 5 (Continued)

No. of Test	Size of Stone	Method of Filling	Wt. of Stone lb.	Wt. of Stone and Water, lb.	Wt. of Water lb.	Vol. of Water cu. ft.	Per cent of Voids	
							By Pouring in Water	From Specific Gravity
	By Use of Vessel Containing 0.694 cu. ft.							
17	$\frac{3}{8}$ -in. Scr.	8 ft. drop	63.00	79.75	16.75	0.27	38.5	45.5
18			63.50	80.25	16.75	0.27	38.5	45.1
							Mean	38.5
19	$2\frac{1}{4}$ -in.- $1\frac{1}{4}$ -in.	8 ft. drop	65.25	85.25	20.00	0.32	45.9	43.6
20			66.50	86.25	19.75	0.31	45.4	42.5
							Mean	45.6

Precautions were taken to prevent absorption of water by the sides of the vessel; and it is believed that there is no possibility of error from this source in the data given in Tables 3, 4, and 5. In some of the experiments the vessel containing the stone was hauled from the chute to the scales on a wagon; and to eliminate a possibility of error in weighing, the team was unhitched while the weight was being taken.

Notice that the first part of Table 3 shows the percentage of voids for the different sizes of stone; while the second shows the variation due to the different methods used in filling the tub. An inspection of the table shows that with each vessel the voids increase with the size of the stone. It also shows that for both vessels the average percentages are fairly uniform, the greatest variation being in the case of the 3-in. (3-in. to 2-in.) stone. In comparing the tests in which the 15- and 20-ft. drops were used, the stone falling 20 feet invariably has a smaller percentage of voids than that falling only 15 feet. The lower part of the table shows that the voids were very materially less for the same size of stone when the tub was filled by the 20-ft. drop, than when the stone was shoveled in. These data show clearly that the density increases with the fall. However, the tests were not sufficient in

number to justify an attempt to deduce a statement of the relation of the height of fall to the density of the mass.

A comparison of the results in the last two columns of Table 3 shows that for screenings the method by pouring in water gives a considerably smaller per cent of voids than by computation, while for the 2-in. (2-in. to $\frac{3}{8}$ -in.) and the 3-in. (3-in. to 2-in.) sizes there is practically no difference by the two methods. Substantially the same conclusions may be drawn from Tables 4 and 5.

Summary of Voids:—A summary of the results in Tables 3, 4 and 5 is given in Table 6.

TABLE 6
SUMMARY OF PER CENT OF VOIDS

Ref. No.	Location of Quarry	Size of Stone	Per Cent of Voids	
			By Pouring in Water	From Specific Gravity
1	Chester	$\frac{3}{8}$ in. Scr.	40.9	46.8
2	"	$\frac{3}{4}$ in. Scr.	43.0	45.6
3	"	2 in. to $\frac{3}{4}$ in.	46.6	46.6
4	"	3 in. to 2 in.	46.1	45.1
5	Joliet	$\frac{1}{2}$ in. Scr.	42.2	47.1
6	"	2 in. to $\frac{1}{2}$ in.	47.9	46.2
7	"	3 in. to 2 in.	47.5	46.1
8	Kankakee	$\frac{3}{8}$ in. Scr.	39.6	46.1
9	"	$1\frac{1}{4}$ in. to $\frac{3}{8}$ in.	45.7	44.7
10	"	$2\frac{1}{4}$ in. to $\frac{3}{8}$ in.	44.3	42.9
11	"	$2\frac{1}{4}$ in. to $1\frac{1}{4}$ in.	46.2	43.4

IV. SETTLEMENT OF CRUSHED STONE IN TRANSIT

Sometimes crushed stone is bought by bulk, in which case it may make a difference whether the volume is measured at the beginning or at the end of the journey. Therefore experiments were made to determine the settlement of crushed stone during transit in wagons and also in railway cars.

Settlement in Wagons:—Observations were first made to determine the relation between the settlement in wagons and the distance hauled. An attempt was made to determine the amount of settlement for regular increments

in the distance hauled. This was done by stopping the team and taking a measurement each successive 100 feet until the settlement for that distance was too small to measure. The measurements in all cases were taken by using two straight edges, one placed across the top of the box and the other resting on the top of the stone. Then as both straight edges were of the same width, each measurement was taken from the top of the upper one to the top of the lower one. Measurements were taken near each side and on the center line, near the front, middle, and back of the load, making a total of nine measurements for each load.

The data for Chester limestone are given in Table 7. The results vary surprisingly,—for example, compare tests No. 2 and 3, or 7 and 8, or 13 and 14. The haul was over about equal distances on macadam, cinders, and earth. The results were obtained within a day or two of each other, and it does not seem possible that the smoothness of the roads could have changed materially in the meantime. An attempt was made to drive equally carefully every time. About the only safe conclusions that can be drawn from these data are: (1) about half of the settlement occurs in the first 100 feet; and (2) the settlement at half a mile is practically the same as that at a mile.

TABLE 7
EFFECT OF DISTANCE HAULED UPON SETTLEMENT IN WAGON
Experiments on Chester Limestone by Mr. Schafmayer

Test No.*	Size of Stone	Method of Loading	Per Cent of Settlement for Hauls of— feet								
			100	200	300	400	500	600	700	2640	5280
3	$\frac{3}{4}$ in. Scr.†	15 ft. drop	7.3	8.3	8.9	9.2	9.5	10.1	10.1	11.2	11.2
4	“	“	5.0	9.7	10.2	10.2	10.4	10.4	10.7	12.4	
6	2 in.— $\frac{3}{4}$ in.	15 ft. drop	2.6	3.7	4.9	5.3	5.3	5.3	5.4	5.4	5.4
7	“	“	5.3	6.2	7.1	7.7	7.9	8.0	8.3	9.2	
9	“	Shoveled	3.5	4.1	4.8	5.3	5.3	5.7	6.5	7.3	
11	3 in.—2 in.	15 ft. drop	0.57	2.6	2.8	4.1	4.25	4.25	4.25	4.9	4.9
12	“	“	3.5	4.2	4.5	4.8	5.0	5.0	5.1	6.0	6.0
14	“	Shoveled	5.0	5.7	6.53	6.53	6.7	6.7	6.7	7.1	7.1

*These numbers refer to the series in Table 8.

†Dusty.

The per cent of settlement of stone from three different localities for a haul of practically one mile is given in Table 8; but the variation for any one size under identically the same conditions (for example, compare the first three lines of the table) is so great as not to warrant any attempt to draw conclusions. It was not possible to secure more accurate data except by an expenditure of time and money much greater than the value of the information seemed to justify.

Settlement in Cars:—The shortage of cars at the time these experiments were made and the desire of the shipper and also of the railway to hurry shipments forward seriously interfered with the scope and value of these

TABLE 8
SETTLEMENT OF CRUSHED STONE IN TRANSIT IN WAGONS

Test No.	Method of Loading	Orig. Vol. cu.yd.	Final Vol. cu.yd.	Per Cent of Settlement	Size of Stone	Dist. Hauled miles	Remarks
Chester Limestone by Mr. Schafmayer							
1	15 ft. drop	1.41	1.23	12.7	$\frac{3}{8}$ -in. Scr.	1	Same for $\frac{1}{2}$ mile
2	15 ft. drop	1.41	1.25	11.4	$\frac{3}{4}$ -in. Scr.	1	Mostly dust
3	"	1.41	1.25	11.4	"	1	Same for $\frac{1}{2}$ mile
4	"	1.41	1.23	12.7	"	1	" " "
			Mean	11.8			Stone dusty, wet
5	15 ft. drop	1.41	1.25	11.4	2-in.- $\frac{3}{4}$ in.	1	Same for 2 miles
6	"	1.41	1.33	5.7	"	1	Same for $\frac{1}{2}$ mile
7	"	1.41	1.28	9.2	"	1	" " "
8	Shoveled	1.41	1.23	12.7	"	1	Same for $\frac{1}{2}$ mile
9	"	1.41	1.31	7.1	"	1	Stone damp
			Mean	9.2			
10	15 ft. drop	1.41	1.27	10.1	3-in.-2 in.	1	A few tailings
11	"	1.41	1.34	4.9	"	1	Same for $\frac{1}{2}$ mile
12	"	1.41	1.32	6.4	"	1	" " "
13	Shoveled	1.41	1.23	12.7	"	1	A few tailings
14	"	1.41	1.31	7.1	"	1	Same for $\frac{1}{2}$ mile
			Mean	8.2			Stone dirty

TABLE 8 (Continued)

Test No.	Method of Loading	Orig. Vol. cu. yd.	Final Vol. cu. yd.	Per Cent of Settlement	Size of Stone	Dist. Hauled miles	Remarks
Kankakee Limestone by Mr. Bowling							
15	*	1.80	1.61	10.6	$\frac{3}{8}$ -in. Scr.	1	Chute at incline of 30°
16	*	1.61	1.46	9.3	"	1	" " "
			Mean	10.0			
17	*	1.80	1.67	7.2	$1\frac{1}{4}$ in.— $\frac{3}{8}$ -in.	1	" " "
18	*	1.61	1.45	9.9	"	$1\frac{1}{2}$	" " "
			Mean	8.6			
Joliet Limestone by Mr. Bowling							
19	4 ft. drop	1.81	1.66	8.3	$\frac{1}{2}$ -in. Scr.	$\frac{1}{2}$	Chute at incline of 45°
20	"	1.86	1.69	9.1	"	$\frac{1}{2}$	" " "
21	"	1.81	1.63	9.9	"	$\frac{1}{2}$	" " "
			Mean	9.1			
22	4 ft. drop	1.81	1.69	6.6	2 in.— $\frac{1}{2}$ -in.	$\frac{1}{2}$	" " "
23	"	1.79	1.67	6.7	2 in.— $\frac{1}{2}$ -in.	$\frac{1}{2}$	" " "
			Mean	6.6			

* Lower end of chute even with top of wagon bed.

experiments. (See Table 9). It will be noticed that the settlement varies greatly for stone of the same size, loaded the same day, and shipped to the same destination on the same train,—for example, compare the second, third and fourth lines of the table. The settlement was measured by the same method as previously described for wagons, and was as carefully determined as possible by that method. Part of the error is doubtless due to a variation in the freedom with which the crushed stone ran out of the loading chute, and to a variation in the details of the method employed in leveling off the load.

TABLE 9
SETTLEMENT OF CRUSHED STONE IN TRANSIT
IN RAILWAY CARS

Test No.	Method of Loading	Original Depth of Load feet	Final Depth of Load feet	Per Cent of Settlement	Size of Stone	Destination	Length of Haul, miles
Joliet Limestone by Mr. Schafmayer							
1	8 ft. drop	2.4	2.4	0.0	$\frac{3}{4}$ -in. Scr.	Springfield	149
2	8 ft. drop	2.4	2.2	8.3	" "	McLean	105
3	" "	2.0	1.75	12.5	" "	"	"
4	" "	2.6	2.3	8.3	" "	"	"
			Mean	9.7			
5	8 ft. drop	2.1	2.08	1.4	2 in.- $\frac{3}{4}$ -in.	Springfield	149
6	" "	2.2	1.9	13.9	" "	"	"
			Mean	7.6			
7	8 ft. drop	2.3	2.1	8.7	" "	McLean	105
8	" "	2.2	1.9	13.9	" "	"	"
9	" "	2.7	2.5	7.4	" "	"	"
10	" "	2.6	2.33	9.7	" "	"	"
11	" "	2.6	2.4	7.7	" "	"	"
			Mean	9.5			
12	8 ft. drop	2.6	2.5	3.8	3 in.-2 in.	Springfield	149
13	" "	2.6	2.3	10.5	" "	"	"
			Mean	7.2			
14	8 ft. drop	2.2	1.95	11.4	" "	McLean	105
15	" "	3.2	3.1	3.4	" "	"	"
16	" "	2.1	2.0	5.0	" "	"	"
17	" "	2.7	2.35	12.9	" "	"	"
18	" "	2.7	2.6	3.7	" "	"	"
19	" "	2.2	2.2	0.0	" "	"	"
20	" "	3.33	3.0	9.0	" "	"	"
21	" "	2.3	2.05	10.8	" "	"	"
22	" "	2.7	2.5	7.4	" "	"	"
23	" "	2.7	2.45	9.2	" "	"	"
24	" "	2.7	2.4	11.1	" "	"	"
25	" "	2.6	2.35	9.6	" "	"	"
26	" "	2.2	2.0	9.1	" "	"	"
			Mean	10.2			

TABLE 9 (Continued)

Test No.	Method of Loading	Original Depth of Load feet	Final Depth of Load feet	Per Cent of Settlement	Size of Stone	Destination	Length of Haul, miles
Joliet Limestone by Mr. Bowling							
1	8 ft. drop	2.37	2.17	8.4	$\frac{1}{2}$ -in. Scr.	Bloom'ton	91
2	" "	2.52	2.31	8.3	2 in.- $\frac{1}{2}$ -in.	"	"
3	" "	2.80	2.62	6.4	" "	"	"
			Mean	7.4			
4	" "	2.57	2.37	7.8	3 in.-2 in.	"	"
Chester Limestone by Mr. Schafmayer							
1	15 ft. drop	3.00	2.7	9.5	$\frac{3}{4}$ -in. Scr.	Springfield	180
2	" "	2.75	2.4	12.5	" "	"	"
3	" "	2.50	2.25	9.8	" "	"	"
			Mean	10.6			
4	Barrows	2.67	2.58	3.4	3 in.-2 in.	"	"
5	15 ft. drop	3.00	2.7	9.5	" "	"	"
6	Barrows	2.58	2.33	8.2	" "	"	"
			Mean	7.0			
Kankakee Limestone by Mr. Bowling							
1	8 ft. drop	2.27	2.15	5.4	2 $\frac{1}{4}$ in.- $\frac{3}{8}$ in.	Bloom'ton	86

It is probable that part of the difference is due to the difference in the care employed in switching the car from the loading chute. At Joliet the cars were switched about a mile from the crusher to the yards in the city, to be weighed; and at the time they were weighed a casual examination was made of the settlement, and the conclusion was drawn that from $\frac{1}{3}$ to $\frac{1}{2}$ of the total settlement took place while the cars were being switched. The cars were continually being moved while they were in the yard, and

hence more accurate observations could not be made as to the effect of switching upon the settlement. The distance from Joliet to McLean is 105 miles, and apparently a further haul of 44 miles to Springfield did not materially increase the settlement. The great variations in results obtained under seemingly like conditions make it unwise to attempt to draw any conclusions. Apparently more tests must be made before any reliable conclusion can be stated concerning the total amount of the settlement or the law of its variation.

The depth of load in Table 9 is the mean of nine separate measurements, and was computed to the nearest hundredth of a foot although it is recorded only to the nearest tenth. The more accurate values were employed in computing the per cent of settlement. However, to eliminate any possibility of error in the arithmetical work, the per cent of settlement was computed to a greater number of places than is justified by the data. A similar statement applies to several of the tables in the subsequent parts of this paper.

Summary of Data on Settlement:—A summary of the data in Tables 8 and 9 is given in Table 10.

TABLE 10
SUMMARY OF DATA ON SETTLEMENT

Ref. No.	Location of Quarry	Size of Stone	Settlement after a Haul of	
			$\frac{1}{2}$ mile or more in wagons	75 miles or more in cars
1	Chester	$\frac{3}{8}$ -in. Scr.	12.7	
2	"	$\frac{3}{4}$ " "	11.8	10.6
3	"	2 in.- $\frac{3}{4}$ in. Scr.	9.2	
4	"	3 in.-2 in. "	8.2	7.0
5	Joliet	$\frac{1}{2}$ in. Scr.	9.1	8.4
6	"	$\frac{3}{4}$ " "		9.7
7	"	2 in.- $\frac{1}{2}$ in. Scr.	6.6	7.4
8	"	2 in.- $\frac{3}{4}$ " "		9.5
9	"	3 in.-2 in.		7.8
10	Kankakee	$\frac{3}{8}$ in. Scr.	10.0	
11	"	1 $\frac{1}{4}$ in.- $\frac{3}{8}$ in. Scr.	8.6	
12	"	2 $\frac{1}{4}$ in.- $\frac{3}{8}$ " "		5.4

V. WEIGHT PER CUBIC YARD OF CRUSHED LIMESTONE

Broken stone is usually sold by weight even though the unit is nominally the cubic yard, since it is the custom to determine the number of cubic yards in a shipment by weighing the shipment and dividing the total weight by the supposed weight of a cubic yard. It does not appear that any adequate observations have been made to determine the weight of a unit of volume of the different sizes and kinds of crushed stone.

Tests to determine the weight of a unit of volume of crushed limestone were made on stone from Joliet, Kankakee, and Chester, both in wagons and in cars, at the same time the record was taken of the settlement, as previously described.

Before beginning to load a car, measurements were taken from a straight edge laid on top of the car body to the floor of the car. These measurements were taken on each side of the car and at the center transversely, and at each end and the middle longitudinally. The stone was loaded into the cars by means of a chute in the bottom of the bin. After the car was loaded the upper surface was leveled off, and the depth of the stone below the top of the car body was determined by measuring down from a straight edge across the top of the car to a similar straight edge lying on the crushed stone. From the above measurement the volume of the stone was computed.

The cars were then switched to the scale track where they were weighed by a representative of the National Weighing Association, each weight being verified by either Mr. Schafmayer or Mr. Bowling. From these data the weight per cubic yard of the loose stone was computed. Measurements similar to those made at the crusher were taken when the car reached its destination; and the weights per unit of volume of the stone when compacted were computed as before.

The data and results of the observations on Joliet, Chester, and Kankakee stone are given in Tables 11, 12 and 13 respectively. For car loads, the "original weight" is after the car was switched about a mile, and the "final weight" is after being shipped 75 miles (a greater distance makes practically no difference); and for wagon loads the weights are at the loading bin and after being hauled a half mile or more.

TABLE 11

WEIGHT PER CUBIC YARD OF JOLIET LIMESTONE

Test No.	Size of Stone	Wt. of Stone, lb.	Original No. of cu. yd.	Final No. of cu. yd.	Original Wt., lb. per cu. yd.	Final Wt. lb. per cu. yd.
Car Loads by Mr. Schafmayer						
1	$\frac{3}{4}$ -in. Scr.	94500	36.0	36.0	2625	2625
2	"	75500	28.9		2612	
3	"	94400	34.6	31.7	2730	2980
4	"	54700	20.9	18.2	2610	3000
5	"	96900	36.2	33.2	2680	2920
				Mean	2652	2881
6	2 in.- $\frac{3}{4}$ -in.	78700	30.6	30.2	2570	2600
7	"	69700	31.5	27.1	2210	2570
8	"	58500	24.8	22.7	2360	2580
9	"	54300	23.6	20.3	2300	2680
10	"	67500	31.0	29.1	2180	2320
11	"	82600	37.5	34.0	2200	2430
12	"	84400	37.5	34.8	2250	2430
				Mean	2296	2516
13	3 in.-2 in.	92400	36.6	35.2	2520	2620
14	"	52700	23.0		2290	
15	"	102300	42.0	40.6	2440	2620
16	"	78600	31.8		2470	
17	"	63050	25.2	23.6	2500	2670
18	"	66600	27.7	24.3	2380	2740
19	"	88600	38.6	37.4	2300	2370
20	"	75800	31.6	31.6	2400	2400
21	"	94800	41.4	37.7	2290	2520
22	"	69000	30.0	26.4	2300	2610
23	"	87900	38.8	36.0	2270	2440
24	"	88300	38.8	35.4	2275	2490
25	"	86900	38.8	34.8	2240	2500
26	"	72800	32.2	28.8	2260	2530
27	"	88200	36.5		2420	
28	"	91000	37.2		2426	
29	"	91300	37.5	33.9	2430	2690
30	"	76800	31.3	28.5	2450	2700
				Mean	2370	2564

TABLE 11 (Continued)

WEIGHT PER CUBIC YARD OF JOLIET LIMESTONE

Test No.	Size of Stone	Wt. of Stone, lb.	Original No. of cu. yd.	Final No. of cu. yd.	Original Wt. lb. per cu. yd.	Final Wt. lb. per cu. yd.
Car Loads by Mr. Bowling						
31	$\frac{1}{2}$ -in. Scr.	83800	31.52	28.85	2659	2905
32	2 in.- $\frac{1}{2}$ -in.	82800	34.40	31.82	2407	2602
33	"	79600	33.64	30.82	2366	2583
				Mean	2386	2592
34	3 in.-2 in.	58300	26.47	24.37	2202	2392
Wagons Loaded by Mr. Bowling						
35	$\frac{1}{2}$ -in. Scr.	4195	1.81	1.66	2318	2527
36	"	4260	1.86	1.69	2290	2521
37	"	4165	1.81	1.63	2301	2555
				Mean	2303	2533
38	2 in.- $\frac{1}{2}$ -in.	4185	1.81	1.69	2312	2476
39	"	4150	1.79	1.67	2318	2485
				Mean	2315	2480

It will be noticed that there is considerable variation in both the original and the final weight per cubic yard. Notwithstanding the variation it is believed that the number of observations is so great as to make the means reasonably reliable; but the table shows that the maximum error of any one observation may be as much as 10 per cent, and hence great accuracy can not be expected from a single observation. Similar results are shown for Chester stone,—see Table 12. There are three errors that affect these results: (1). Errors in determining the value of the stone at the quarry and at the destination. (2). Errors in the weight of the car as stenciled upon it. The original weight may have been de-

TABLE 12
WEIGHT PER CUBIC YARD OF CHESTER LIMESTONE

Test No.	Size of Stone	Wt. of Stone lb.	Original No. of cu. yd.	Final No. of cu. yd.	Original Wt. lb. per cu. yd.	Final Wt. lb. per cu. yd.	Remarks
Car Load by Mr. Schafmayer							
1	$\frac{3}{4}$ -in. Scr.	109100	41.97	38.0	2600	2870	Damp
2	"	70600	28.14	24.6	2509	2870	
3	"	92900	36.72	33.1	2530	2810	
				Mean	2546	2850	
4	3 in.-2 in.	96500	41.60	38.2	2320	2530	Hand made Damp
5	"	81500	32.91	31.8	2476	2560	
				Mean	2398	2545	
6	3 in.-2 in.	106100	41.97	38.0	2528	2790	Wet
Wagon Loads by Mr. Schafmayer							
7	$\frac{3}{4}$ -in. Scr.	3550	1.41	1.23	2518	2886	15 ft. drop
8	"	3550	1.41	1.25	2518	2840	15 ft. drop
9	"	3460	1.41	1.25	2450	2770	15 ft. drop
10	"	3420	1.41	1.23	2425	2780	15 ft. drop
11	"	2430	1.00		2430		No haul, 15 ft. drop
12	"	2395	1.00		2395		No haul, 15 ft. drop
13	"	2435	1.00		2435		No haul, 15 ft. drop
				Mean	2453	2819	
14	2 in.- $\frac{3}{4}$ -in.	3360	1.41	1.28	2380	2625	15 ft. drop
15	"	3250	1.41	1.23	2305	2642	Shoveled
16	"	3460	1.41	1.33	2450	2600	15 ft. drop
17	"	3200	1.41	1.31	2270	2445	Shoveled
18	"	2375	1.00		2375		No haul, 15 ft. drop
19	"	2320	1.00		2320		No haul, 15 ft. drop
20	"	3250	1.41	1.25	2305	2600	No haul, 15 ft. drop
				Mean	2444	2582	
21	3 in.-2 in.	3200	1.41	1.23	2270	2601	Shoveled
22	"	3330	1.41	1.33	2360	2505	15 ft. drop
23	"	2390	1.00		2390		No haul, 15 ft. drop
24	"	3480	1.41	1.34	2470	2595	15 ft. drop
25	"	3290	1.41	1.31	2335	2510	Part dirt, shoveled
26	"	2370	1.00		2370		No haul, 15 ft. drop
27	"	3350	1.41	1.27	2376	2638	15 ft. drop
				Mean	2367	2570	

terminated when the car was wet; while in the observations under consideration the cars were dry. (3). In weighing a string of cars, either empty or loaded, there is some error due to the action of the coupler.

Table 11 shows that the weight per cubic yard of screenings is more than that of coarser stone, but also shows that 3-in. stone weighs more per cubic yard than 2-in. Similar results obtained for wagon loads, and also for Chester and Kankakee stone, both for car loads and for wagon loads—see Tables 12 and 13.

TABLE 13

WEIGHT PER CUBIC YARD OF KANKAKEE LIMESTONE.

Test No.	Size of Stone	Weight of Stone, lb.	Original No. of cu. yd.	Final No. of cu. yd.	Original Wt. lb. per cu. yd.	Final Wt. lb. per cu. yd.	Remarks
Car Load by Mr. Bowling							
1	2½ in.-¾ in.	62900	27.83	26.32	2260	2390	
Wagon Loads by Mr. Bowling							
2	¾ in. Scr.	4085	1.80	1.61	2270	2537	
3	“	4170	1.61	1.46	2590	2856	
				Mean	2430	2697	
4	1¼ in.-¾ in.	3840	1.80	1.67	2133	2299	
5	“	4050	1.61	1.45	2516	2793	
				Mean	2325	2546	

Summary of Weights:—Taking an average of the preceding results for each size of stone from each quarry the summary shown in Table 14 is obtained.

TABLE 14

SUMMARY OF WEIGHTS OF CRUSHED LIMESTONE

Results in Pounds per Cubic Yard

Ref. No.	Location of Quarry	Size of Stone	Wagon Loads		Car Loads	
			Wt. at Crusher	After a Haul of $\frac{1}{2}$ mile or more	Wt. at Crusher	After a Haul of $\frac{1}{5}$ miles or more
1	Joliet	$\frac{1}{2}$ -in. Scr.	2303	2533	2659	2905
2	"	$\frac{3}{4}$ -in. Scr.			2652	2882
3	"	2 in.- $\frac{1}{2}$ -in.	2315	2480	2386	2592
4	"	2 in.- $\frac{3}{4}$ -in.			2296	2516
5	"	3 in.-2 in.			2361	2553
6	Chester	$\frac{3}{4}$ -in. Scr.	2442	2797	2546	2850
7	"	2 in.- $\frac{3}{4}$ -in.	2344	2582		
8	"	3 in.-2 in.	2367	2569	2348	2545
9	Kankakee	$\frac{3}{8}$ -in. Scr.	2430	2697		
10	"	$1\frac{1}{4}$ in.- $\frac{3}{4}$ -in.	2325	2546		
11	"	$2\frac{1}{4}$ in.- $\frac{3}{8}$ -in.			2260	2390

Relations between Actual and Nominal Weight of Crushed Stone.—

As is well known, it is the universal custom to load a car more than its rated capacity; and similarly it seems to be the custom of laborers when loading a car with crushed stone, to put in more than directed. This fact causes an erroneous idea of the weight of a yard of the material among the railway officers, as they weigh the car and divide the weight of the stone by the nominal number of yards to obtain the weight per cubic yard. Since the actual volume is not measured, the number of yards is taken from the bill of lading submitted by the shipper, which is approximate and is usually too small; and consequently the weight per cubic yard derived by this method is usually somewhat too great. For example, the Superintendent of the Wabash, Chester and Western Railway weighed a large number of cars of stone at Chester, and obtained by this method weights of 2600 pounds and over per cubic yard. In all his observations the number of yards was taken as given on the bills.

To determine the relation between the actual weight of crushed stone and the weight found as above, accurate measurements of ten cars being loaded at Chester were made by Mr. Schafmayer to ascertain if the high weights per cubic yard obtained by the railroad were due to overloading. The results of these tests are shown in Table 15. In every case the actual contents of the car are greater than the number of yards in the bill. The average excess is 1.71 yards for an average nominal load of 26 yards, an average excess of 6.6 per cent. This gives an apparent average weight of 2558 pounds for a yard actually weighing only 2400 pounds. It can be readily seen that under such conditions, it is not surprising that railway officials have an exaggerated idea as to the weight of a cubic yard of crushed stone.

TABLE 15
EXCESS OF ACTUAL LOADING IN CARS OVER BILLING

Ref. No.	No. of Yds. Named in Bill of Lading	Actual Number of Cu. Yd.	No. of Yds. in Excess of the Billing	Per Cent of Excess	Actual Wt., lb. per cu. yd.	Apparent Wt., lb. per cu. yd.
1	25	25.7	0.7	2.8	2420	2488
2	25	25.7	0.7	2.8	2420	2488
3	25	27.7	2.7	10.8	2420	2684
4	25	27.7	2.7	10.8	2420	2684
5	25	27.7	2.7	10.8	2420	2684
6	25	25.8	0.8	3.2	2420	2510
7	30	32.0	2.0	6.7	2420	2583
8	25	25.7	0.7	2.8	2420	2488
9	30	31.4	1.4	4.7	2420	2534
10	25	27.7	2.7	10.8	2420	2684
Av.	26	27.71	1.71	6.6	2420	2581

VI. COEFFICIENTS FOR DETERMINING WEIGHT OF CRUSHED STONE

In the introduction it was suggested that possibly coefficients could be determined by which to deduce the weight per unit of volume of crushed stone when the weight of a unit of solid stone or the specific gravity was known. Table 16 shows such coefficients for the various sizes for three kinds of stone, at the crusher and also at the destination, both in cars and in wagons. The

TABLE 16

COEFFICIENTS BY WHICH TO DETERMINE THE WEIGHT IN POUNDS
PER CUBIC YARD OF CRUSHED LIMESTONE

Kind of Stone	Size of Stone	Having the Weight of a Cu. Ft. of Solid Stone			Having the Weight of a Cu. Yd. of Solid Stone			Having the Specific Gravity		
		Wt. of Solid Stone, lb. per cu. ft.	Coeffi- cient		Wt. of solid stone, lb. per cu. yd.	Coefficient		Sp. Gr. of Stone	Coefficient	
			Car	Wagon		Car	Wagon		Car	Wagon
		Weight at Crusher								
Chester	$\frac{3}{4}$ -in. Scr.	160.4	15.9	15.3	4331	0.588	0.566	2.57	990.7	954.5
Joliet	$\frac{3}{4}$ -in. Scr.	169.1	15.7	13.6	4566	0.582	0.504	2.71	981.2	849.8
"	$\frac{1}{2}$ -in. Scr.	169.1	15.4		4566	0.572		2.71	963.8	
Kankakee	$\frac{3}{8}$ -in. Scr.	162.8		14.9	4397		0.553	2.61		931.0
Mean	$\frac{1}{2}$ -in. Scr.	165.4	15.7	14.6	4465	0.581	0.541	2.65	978.6	911.8
Kankakee	$1\frac{1}{4}$ in.- $\frac{3}{8}$ in.	162.8		14.3	4397		0.529	2.61		890.8
Chester	2 in.- $\frac{3}{4}$ -in.	160.4		14.6	4331		0.541	2.57		912.1
Joliet	2 in.- $\frac{1}{2}$ -in.	169.1	14.1	13.7	4566	0.523	0.507	2.71	880.4	854.2
"	2 in.- $\frac{3}{4}$ -in.	169.1	13.6		4566	0.503		2.71	847.2	
Kankakee	$2\frac{1}{4}$ in.- $\frac{3}{8}$ in.	162.8	13.9		4397	0.514		2.61	865.9	
Mean	2 in.- $\frac{1}{2}$ in.	165.4	13.9	14.2	4465	0.513	0.524	2.65	864.5	883.2
Chester	3 in.-2 in.	160.4	14.9	14.8	4331	0.554	0.546	2.57	933.1	921.0
Joliet	3 in.-2 in.	169.1	14.0		4566	0.517		2.71	871.2	
Mean	3 in.-2 in.	164.8	14.4	14.8	4448	0.536	0.546	2.64	902.2	921.0
		Weight at Destination								
Chester	$\frac{3}{4}$ -in. Scr.	160.4	17.8	17.6	4331	0.658	0.651	2.57	1108.9	1096.9
Joliet	$\frac{1}{2}$ -in. Scr.	169.1	17.2	15.0	4566	0.636	0.555	2.71	1072.0	934.7
"	$\frac{3}{4}$ -in. Scr.	169.1	16.9		4566	0.625		2.71	1053.5	
Kankakee	$\frac{3}{8}$ -in. Scr.	162.8		16.6	4397		0.613	2.61		1033.3
Mean	$\frac{1}{2}$ -in. Scr.	165.4	17.3	16.6	4465	0.640	0.606	2.65	1078.1	1021.6
Kankakee	$1\frac{1}{4}$ in.- $\frac{3}{8}$ in.	162.8		15.6	4397		0.579	2.61		975.5
Chester	2 in.- $\frac{3}{4}$ -in.	160.4		16.1	4331		0.596	2.57		1004.7
Joliet	2 in.- $\frac{1}{2}$ -in.	169.1	15.3	14.7	4566	0.568	0.543	2.71	956.5	915.1
"	2 in.- $\frac{3}{4}$ -in.	169.1	14.9		4566	0.551		2.71	928.4	
Kankakee	$2\frac{1}{4}$ in.- $\frac{3}{8}$ in.	162.8	14.7		4397	0.544		2.61	915.7	
Mean	2 in.- $\frac{1}{2}$ in.	165.4	15.0	15.4	4465	0.554	0.570	2.65	933.5	959.9
Chester	3 in.-2 in.	160.4	15.9	16.0	4331	0.588	0.593	2.57	990.2	1000.0
Joliet	3 in.-2 in.	169.1	15.1		4566	0.559		2.71	942.0	
Mean	3 in.-2 in.	164.8	15.5	16.0	4448	0.574	0.593	2.64	966.1	1000.0

means are stated in Table 16 in such a manner as to show the average result for each size.

Disregarding whether the stone is measured in a car or a wagon, and also disregarding whether it is measured at the crusher or at its destination, the following summary of Table 16 is obtained.

SIZE OF STONE	MEAN COEFFICIENT BY WHICH TO MULTIPLY THE WEIGHT OF A CUBIC FOOT OF SOLID LIMESTONE TO OBTAIN THE WEIGHT OF A CUBIC YARD OF THE CRUSHED STONE
$\frac{1}{2}$ -in. screenings.....	15.5
2 in. to $\frac{1}{2}$ inch.....	14.6
3 in. to 2 inch.....	15.2
	15.1

Notice that the coefficient is largest for the finest stone, and smallest for the intermediate size. The same is true for trap (see Table 17) even though the sizes slightly differ. This seems to prove that the weight of screenings is greater than that of coarser stone, while the weight of the intermediate size is less than that of either extreme size.

APPENDIX I

WEIGHT OF VOIDS OF CRUSHED TRAP

A careful search has been made of engineering literature, and below is the only definite information discovered.

In the Journal of the Association of Engineering Societies, Volume 11 (1892), page 424, W. E. McClintock, at present Chairman of the Massachusetts Highway Commission, gives an account of six experiments made by him to determine the weight of a unit of volume of crushed trap. In the first experiment he weighed the contents of a bin holding $29\frac{1}{8}$ cubic yards,* and found the weight of stone that had passed a $\frac{1}{2}$ -inch screen to be 2605 pounds per cubic yard, and in another test under the same conditions, to be 2690 pounds; and when the broken stone was wet the weight was 2480 pounds per cubic yard. In another experiment he weighed the stone in a bin holding 89.8 cubic yards, and found the weight of stone that had passed a $1\frac{1}{2}$ -inch screen and had been caught on a $\frac{1}{2}$ -inch screen to be 2423 pounds per cubic yard. In a third experiment he weighed the stone in a bin containing 89.7 cubic yards, and found the weight of the stone that had passed a 3-inch screen and had been caught on a $1\frac{1}{2}$ -inch screen to be 2522 pounds per cubic yard. He also measured six cars and weighed the contents, and found the weight of the last mentioned size to be 2531 pounds per cubic yard. The following statement shows the relative proportions of the several sizes of crushed trap.

SIZE OF STONE	PER CENT
$\frac{1}{2}$ -inch screenings	13.24
$1\frac{1}{2}$ inch to $\frac{1}{2}$ -inch.....	23.89
3 inch to $1\frac{1}{2}$ inch.....	62.87
Total output of crusher.....	100.00

From the weight per cubic foot of solid stone given by Mr. McClintock and the above weights of the broken stone, the per cent of voids was computed. A summary of Mr. McClintock's experiments is given in Table 17, and the coefficients for trap are given in Table 18.

*Mr. McClintock privately informed the writer that the average drop of the stone into the bins was about 8 feet.

TABLE 17

WEIGHT AND VOIDS OF CRUSHED TRAP

Ref. No.	Size of Stone	Weight in lb. per cu. yd.	Per Cent of Voids, computed
1	$\frac{1}{2}$ -in. Screenings, in bin, dry	2605	
2		2690	
	Mean	2648	46.5
3	in bin, wet	2480	
4		2432	
5	$1\frac{1}{2}$ in. to $\frac{1}{2}$ in. in bin	2522	50.2
6		2531	
	Mean	2526	48.1

TABLE 18

COEFFICIENTS BY WHICH TO DETERMINE THE WEIGHT IN POUNDS PER CUBIC YARD OF CRUSHED TRAP

Ref. No.	Size of Stone	Having the Weight of a Cubic Foot of Solid Stone		Having the Weight of a Cubic Yard of Solid Stone		Having the Specific Gravity	
		Wt. of Sol. Stone lb. per cu. yd.	Coefficient	Wt. of Sol. Stone lb. per cu. yd.	Coefficient	Specific Gravity	Coefficient
1	$\frac{1}{2}$ -in. screenings	180.7	14.6	4879	0.541	2.90	914.4
2	$1\frac{1}{2}$ in.- $\frac{1}{2}$ in.	180.7	13.5	4879	0.500	2.90	839.7
3	3 in.- $1\frac{1}{2}$ in.	180.7	13.9	4879	0.515	2.90	872.2
	Mean		13.7		0.519		875.4

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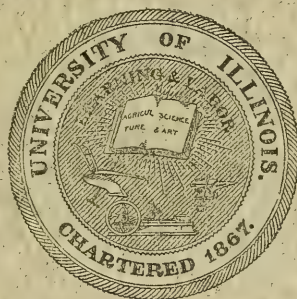
THE MODIFICATION OF ILLINOIS
COAL BY LOW TEMPERATURE
DISTILLATION

BY

S. W. PARR

AND

C. K. FRANCIS



UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

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UNIVERSITY OF ILLINOIS

ENGINEERING EXPERIMENT STATION

BULLETIN No. 24

JUNE 1908

THE MODIFICATION OF ILLINOIS COAL BY LOW TEMPERATURE DISTILLATION

BY S. W. PARR, PROFESSOR OF APPLIED CHEMISTRY

AND

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I. INTRODUCTION¹

In the consumption of fuel for industrial purposes, two fundamental considerations must be kept in mind: first, there must be economy in the use of material; second, there must be regard for the comfort and health of the community. Modern practice has made marked advances in recent years in the observance of the first consideration. As might be expected, the initiative in any matter involving economy comes largely from the user, the impulse being the very natural one of self-interest. For results under the second consideration, obligatory measures, in the main, have predominated, the pressure coming from without by mandate rather than from within by preference, and advancement has been at a slower rate. Any study of conditions or of material which will promote advancement along either line is important. The present paper is a preliminary report of a series of experiments carried on in the Chemical Department of the University of Illinois, more or less continuously since 1902, having in mind primarily such a change in the chemical composition of coal as would modify or minimize the production of smoke. The facts developed thus far are mainly of scientific interest but they are such as must necessarily precede any technical study of the processes contemplated.

¹Mainly taken from the thesis of Deane Burns, B. S., 1907, University of Illinois.

Concerning the waste of the fuel supply, there are broader reasons for economy than the mere selfish demands of the individual or corporation interested. Notwithstanding the fact that the coal fields of the United States are extensive, the coal supply is not inexhaustible. Campbell¹ states that at the present rate of increase, the available coal fields of this country will be used up in ninety years, but if the rate of consumption of 1905 remains constant the fields will last four thousand years. It is hardly possible that the great activity which has so characterized the past twenty-five years, in the building of railroads, steamship lines, and factories will be as marked in the future as it has been in the past. There will be, however, a constant increase in the consumption of coal unless some new source of power is developed. It does not seem probable that the rate of increase will be materially affected for a great many years to come; hence, Campbell estimates that four hundred years will be nearer the truth than four thousand.

The sources of loss are mainly two; one is found in wasteful methods used in the mining, and the other in the combustion of coal. Although it is not within the province of this work to discuss the present day methods of mining, it might be well to mention the fact that from 40 to 60 per cent of the coal is left in the mine in the form of walls, pillars, roofs, dust, etc. In the matter of economic combustion, the advancement has been almost entirely on the mechanical side as illustrated in the modern stokers, improved combustion chambers, the sizing and washing of coal, etc.

The idea of the modification of the chemical form of the fuel as a preparation for greater efficiency has been of slow development, but it is well illustrated in the transformation of lignites into the gaseous form for use in internal combustion engines. If it were possible to apply the same method to changing the form of coals of the bituminous type and maintain a corresponding improvement in efficiency, a great advance would be made both on the side of economy and also as related to a more sanitary and comfortable state of the atmosphere, which would result from the absence of smoke. A chemical modification of quite a different type is contemplated in this work, viz., a modification of the volatile matter either in amount or form, so that a smokeless fuel

¹National Geo. Mag., Vol. 18, 1907.

will result and possibly also a fuel of a higher rate of efficiency in combustion.

A study of the development of coals in their various forms seems to indicate that certain natural agencies have been active, which might be made to accomplish similar results when artificially applied. The basis for some such theory may be more readily understood from a review of the various discussions of (a), coal formation, and (b), coal structure, as given below.

Formation of Coal.—Campbell¹ advances the hypothesis that the transformations of the vegetable matter into the various grades of coal are accomplished in a large measure by three factors: time, heat and crustal movements. Campbell shows that the element of time is not as important as the other two, since vegetable matter submitted to the action of molten lava, for instance, very likely undergoes rapid alteration into anthracite or natural coke, because the escaping gases are the lightest hydrocarbons, while the remaining material is the heavier or fixed carbon. Upon the other hand, if the heat is only a gradual one, or if the surrounding rocks are unbroken, the process is a slow one, as the heat is either not sufficient, or the products of distillation are not able to find a ready means of escape.

Continuing, the writer states that a study of the crustal movements does not justify the conclusion that coal was produced by distillation due to the heating effect of such movements. If, for instance, the folding of the rocks into great synclines and anticlines has changed the coal into anthracite in eastern Pennsylvania, why has not the same amount of movement in some of the isolated synclines of Pocono rocks in Maryland and Virginia produced the same result? It has not done so, and, therefore, the change to anthracite does not seem to be due alone to earth movements.

The action of heat, of course, produces fractional distillation, but whether this takes place rapidly or slowly, at a high or low temperature, the process is regulated by the surrounding or local conditions, and, since these control the process of metamorphism, they are of equal importance with the original cause. The nature of the residual product, or coal, is largely dependent upon these

¹Economic Geology, Vol. 26, 1905.

U. S. G. S. Prof. Paper, No. 48, 1906.

Bulletin Amer. Inst. Min. Eng., March, 1906.

modifying conditions. In the first place, the extent of the change depends upon the readiness with which the gases can escape. If these are liberated as soon as they are formed, the rate of change depends upon the amount of heat applied. If, however, the vegetable matter is held between impervious layers of rock, and under great pressure, the gases can not form, and consequently, there is but little change, despite the fact that considerable heat is applied.

Campbell gives the following résumé:

1. The change from peat to lignite, from lignite to bituminous, and from bituminous to anthracite, is a process of fractional distillation due to heat.

2. The heat may be applied locally and with great intensity, as in the case of volcanic action, or it may be imperceptible, but applied throughout a long period of time.

3. In the latter case the action is so slow, and of such a nature, that it is controlled largely by conditions which accelerate or retard the process.

4. The principal conditions controlling the distillation are the porosity or the impermeability of the rocks permitting or retarding the escape of the gases formed during the process.

5. Porosity may be due either to coarseness of grain or to fissures. Where great masses are concerned, the former has little or no effect, as the rocks are practically impervious; therefore, the great controlling condition in coal metamorphism is the number of fissures. These are produced by joints and by cleavage; and where fissures are found, the coal is in a high degree of carbonization; where they are absent, it is changed only slightly from its original peaty condition.

Cross and Bevan¹, from a study of the action of sulphuric acid upon cellulose and other vegetable fibres, as well as upon coal itself, advanced the opinion that there are two possibilities as to the formation of coals. (1) The lignification of structures originally consisting of pure cellulose; (2) Such transformation may have resulted from the combination of cellulose and aromatic bodies, formed elsewhere in the plant and probably as a residue from the oxidation of hydrocarbons. Reasoning from the investigations, they have assigned to lignocellulose a formula, $C_{12}H_{18}O_9$.

¹Phil. Mag. 352; 1882.

Jour. Amer. Chem. Soc., 55, 199; 1889.

Parr¹, from the study of cellulose and coal, reasons that during the process of decomposition, cellulose may undergo a transformation, producing lignite, ($C_{30}H_{22}O_4$); bituminous coal, ($C_{22}H_{26}O_3$); and semi-bituminous coal, ($C_{30}H_{16}O$). It may be reasonable to believe that if these various forms of coal were produced from cellulose and similar substances, and are more or less closely related, artificial methods may be devised for transforming the one into the other. While these coal molecules are, of course, hypothetical, they have their counterparts in the actual constitution of coal as shown by analyses.

Each of these theories in regard to the formation of coal offers suggestions for the production of similar results artificially. Thus, the work of Campbell shows that under conditions which permit the gases to escape, heat is of vital importance. Those of Parr show, to a certain extent, how the transition from the lower forms to the higher ones is accomplished. Given, then, the conditions favorable for the escape of the gases and heat in the correct amount, it would seem that the results of nature might be duplicated. The work of Cross and Bevan shows the influence of the original material from which the coal was produced. The character of the original vegetable matter and the different conditions under which the coal was formed, yield various results in the final products. Not all anthracite coals are alike, nor are all bituminous coals the same. Different coals, treated under identical conditions, behave in a variety of ways. These variations in the different kinds of coal can be explained only by the pure chemistry of the same.

The Constitution of Coal.—Within the last twenty-five years numerous attempts have been made to prove that coal is a definite chemical compound, or a mixture of compounds which are similar. Many chemists and investigators have succeeded in separating from coal by means of such extracting agents as ether, chloroform, benzene, pyridine, as well as by acids and alkalies, various products which show to some extent that the proximate ingredients are similar in various coals.

As early as 1805 Hatchett² published the results obtained from an investigation of the action of nitric acid upon coal. In

¹Ill. Geol. Surv. Bulletin No. 3; 1906.

Jour. Amer. Chem. Soc. 28, 1425; 1906.

²Phil. Mag. 1805.

1879 E. Guignet¹ found that after drying coal for an hour at 110° C., and powdering it to the finest possible state, he could extract as much as four per cent with boiling phenol. The filtrate from the above was brown in color, and deposited brown flakes from the solution upon the addition of alcohol. The precipitated substance was attacked with difficulty by nitric acid, forming a yellow product. Repeated treatment of the original coal with nitric acid, filtering, evaporating the filtrate to dryness, taking the residue up with water, adding barium carbonate, filtering and removing the barium with sulphuric acid, gave a solution containing tri-nitro-resorcinol, (oxy-picric acid); oxalates were also found in the solution. Guignet believed that all coals contained a resin-like substance which, when treated with nitric acid, produced oxy-picric acid. He also found that coal contained a cellulose-like substance which was attacked by alkaline hydroxides at temperatures above 100° C.

The action of alkalis upon coals was further studied by P. F. Reinsch² in 1885. From a series of experiments the author comes to the conclusion that coals are composed of two substances which are characterized by their action towards alkali. That part which can be extracted by means of alkali is composed of "substances which are quite amorphous, do not exhibit signs of organic structure, possess dyeing properties, and are characterized by their resistance to the action of mineral acids". Kramer³ considers coal to be the anhydrides of glycol and glycoic acids and their amido and oxy-derivatives.

R. T. Friswell⁴ pointed out that the effect of dilute (1:2) nitric acid was to convert coal into substances which, upon drying, were "black, lustrous and very brittle, insoluble in water, but very easily soluble in ammonia and other alkali fluids. In general, these products behaved like nitro compounds". If the solutions obtained by dissolving the nitro-compound in alkali are treated with acids, a flocculent, bulky brown precipitate results, which is insoluble in mineral salts, but is soluble to some extent in pure water.

¹Comptes Rendus, 88, 590; 1879.

²Ding. Poly. Jour. 256, 224; 1885.
Jour. Chem. Soc. 876; 1885.

³Fisher's Jahresbericht, 113; 1887.

⁴Proc. Chem. Soc. 9, 8; 1892.

Anderson and Roberts¹ studied the action of dilute nitric acid. The product or "coal acid" was always of a definite composition, even though it might not have been composed of the same chemical individuals. Heating from 300° to 310° C. in carbon dioxide and extracting with nitric acid gives a coal acid of the same composition as the one from raw coal. In a continuation of these studies Anderson² concludes that "whether or not different coals contain bodies that are in a genetic or homologous series, such bodies form, at least, only a part, and it may be but a minor part of the mineral. And, furthermore, that a considerable part of the organic matter consists of a complex compound comparatively rich in nitrogen, and that above all resinous matter is always present to a small, but fairly constant degree".

P. Siepmann³ in 1891 investigated the effects of various solvents such as ether, alcohol, and chloroform upon the resinous constituents of coal. By treating a Westphalian coal of the following composition:

C = 80.31%; H = 5.50%; O and N = 12.94%; S = 1.25%, three products were obtained. That extracted by ether amounted to 0.3 per cent, after the ether extraction; 0.75 per cent more was extracted by means of chloroform; and, finally, 0.25 per cent was removed with alcohol. Two substances, which were identical, were obtained by means of ether, from two different brown coals; a fact which, in the opinion of the author, may justify the conclusion that the two products were identical and individual in body. The following table taken from the above mentioned article, shows the percentage composition of the extracted substances.

Solvents	Alcohol	Chloroform	Ether	Composition of Residue
Per cent extracted	0.25	0.75	0.30	
Carbon	72.52	78.82	78.74	74.00
Hydrogen	10.08	8.56	9.64	4.77
Oxygen	17.40	9.97	11.62	20.09
Sulphur				
Nitrogen		2.65		1.14

¹Jour. Soc. Chem. Ind., 17, 1013; 1898.

²Ibid, p. 1019.

³Ztg. f. Berg. u. Hutten, 36, 26; 1891.

Jour. Soc. Chem. Ind. 10, 753; 1891.

Ibid 10, 147; 1908.

The author further states that while the caking power of bituminous coal is considerably diminished by the above treatment, it is not entirely destroyed even though a perceptible amount of the resinous constituents of the coal is removed. Smith¹ verifies the fact that the caking power is not all destroyed by the treatment with alkali. He also studied the solvent effect of benzine. From a sample of Japanese (Miike) coal, as much as ten per cent was extracted with the above named solvent. The extracted substance was freed from benzine and subjected to fractional distillation. The portion which was distilled at 175° to 200° was naphtha. The portion distilling at 250° to 300° was treated with sodium hydroxide for phenols, then sulphuric acid was added, when a phenol of higher molecular weight was formed, having the odor of pseudocuminol. No evidences of pyridine or quinoline bases were noticed. Upon cooling the 300° fraction, paraffin scale separated out. In conclusion, the author states that the bitumen which was extracted must have come from vegetable matter different from the body of the coal. Further investigations along this line show a South American lignite, 80.1 per cent of which was soluble in benzine.²

Shaw³ succeeded in nitrifying coal by submitting it to the prolonged action of a mixture of concentrated sulphuric and nitric acids. This treatment raised the "volatile matter" of the coal from 27 to 77 per cent. The "nitro coal" was quite soluble in caustic alkali and 11 per cent was soluble in methyl alcohol. Attempts to reduce the nitro coal were not successful. Luzi⁴ considered that coal might be a derivative of graphitic acid.

Cross and Bevan⁵ tried the oxidizing effect of hydrochloric acid and potassium perchlorate. They secured a chlorinated product which was very similar to a product obtained by treating lignitic fibres in an analogous manner. The remaining coal substance was completely soluble in caustic alkali. Smyth⁶ obtained, by the use of the same reagents, products which were "acidic in properties and form a dark brown solution with caustic alkali and ammonia".

¹Jour. Soc. Chem. Ind., 10, 978; 1891.

²Ibid 17, 985; 1898.

³Chem. News, 74, 262; 1896.

⁴Ztg. f. Berg. w. Hutten, 38, 96; 1893.

⁵Phil. Mag. 1882.

⁶Chem. News, 74, 262; 1896.

From the preceding investigations, the following conclusions may be drawn:

1. That coal was not derived from cellulose alone, but from a combination of cellulose and aromatic bodies formed elsewhere in the plant.

2. That the main portion of coal is a definite unit, associated with a small amount of bitumen or resinous matter.

3. That the structure and properties of coal are greatly modified by the removal of certain portions of the coal by means of solvents.

Action of heat.—It has been shown that certain solvents exert a modifying influence upon coal. The action of heat is of even greater influence than that of solvents, and its application to coal will, in general, produce two results:—

1. Removal of occluded gases.

2. Volatilization of the hydrocarbons of the coal itself. To draw an accurate line between these two would be a rather difficult matter as gases are given off continually from the slightest heat treatment to the point of carbonization of the coal.

Thomas¹ showed that coal heated in a vacuum at 100° C. gave off an appreciable quantity of gas. When coal was submitted to prolonged action of heat and vacuo, not only did the quantities of gas increase, but the quality appeared to vary as the reaction progressed. Late in the action, volatile products came over which condensed to crystalline solids, showing that these hydrocarbons did not exist as such in the coal, but were products of the reactions.

Smith² notes that the preheating of coals destroys their caking properties.

Siepmann³ has shown that the caking properties of coal are modified by extraction of the coal with ether and chloroform. The property of "caking" is then due to the easily volatile hydrocarbons. Investigations similar to those of Thomas⁴ were carried on by Bedson.⁵ Eight hundred seventy grams of coal dust were treated in vacuo with the following results. The first treatment was for twenty days at a temperature of 30° C. The results, when tabulated, are as follows:

¹Jour. Soc. Chem. Ind. 144: 1876.

²Ibid 10, 975: 1891.

³Ibid 10, 752: 1891.

⁴Ibid 144: 1876.

⁵Chem. News, 68, 187: 1893.

Time	Temperature	Volume of Gas
1.20 days	30° C.	100.9 cc N. T. P.
2.10 days	50° C.	160.6 cc N. T. P.
3.10 days	60° C.	116.3 cc N. T. P.
4.7 days	60 to 80° C.	286.0 cc N. T. P.
5.9 hrs.	100° C.	89.5 cc N. T. P.

The above amounts were all obtained from the same sample, the process being continued at a higher temperature when all the gas had been given off at the lower one. Upon analysis, the above examples gave the results in the accompanying table:—

	1	2	3	4	5
Carbon Dioxide	5.77	8.34	12.12	27.35	20.80
Oxygen	9.33	7.31	5.35	0.56	4.16
Carbon Monoxide				1.68	2.34
Olefins		0.39	0.77	2.14	4.74
Paraffins	3.16	4.95	9.39	31.86	29.80
Nitrogen	81.60	79.01	72.37	35.70	38.16

No hydrogen is given off and the oxygen content decreases as the temperature increases. Olefins are present in considerable amounts.

F. G. Trobridge¹ obtained similar results, and from a study of freshly hewn coal, came to the conclusion that such coal absorbs gases from the air, showing a particular preference towards oxygen, while at the same time it was giving off its own occluded gases. E. Bornstein² makes a study of the decomposition of coal at relatively low temperatures, that is, below 450° C. His results, briefly stated, show that the amounts of the aqueous distillates were in the same order as the "oxygen numbers" of the original coals. Small amounts of ammonia, hydrogen sulphide, and catechol were found in the distillates, and the residues left after distillation were of more uniform composition than the original coals. The gaseous distillates ranged in composition as follows:—

¹Jour. Soc. Chem. Ind. 25, 1129; 1906.

²Verh. Ges. Deutsch. Ntf. w. Arzte 1904, II Teil. I Haeft 141. See also Chem. Central-Blatt 497; 1906.

1. Heavy hydrocarbons.....5 to 14 per cent
2. Methane and homologues....55 to 76 per cent
3. Hydrogen.....5 to 16 per cent

It is thus seen that the hydrogen is lower, and the hydrocarbons higher than in ordinary illuminating gas.

The work of Bornstein, which shows that the aqueous distillates are in the same order as the oxygen numbers, leads to a discussion of the nature of water or moisture in coal. It is a well known fact that moisture in coal is a varying quantity. Campbell,¹ in discussing this, says, "This is a phase of the subject upon which there is but little available information, but enough has been done to show that coal in a finely divided state and at ordinary temperatures is very susceptible to change in atmospheric conditions, not only in regard to moisture content, but also to volatile hydrocarbons". If the water retained by an air-dried sample of coal be driven off by heat and the dried sample afterwards exposed to an atmosphere of moist air, the moisture is, to a large extent, reabsorbed at a very rapid rate. Thomas², in reviewing the work of Dr. Hoffman of the Canadian Geological Survey, says that part of the water in coal is very obstinately held and there is no doubt that it is in chemical combination. The water is certainly given off as such, but it is not definitely known whether the hydrogen is in direct union with the oxygen or in combination with the carbon. Cross and Bevan³ say upon this subject that some of the carbon is doubtless in combination with the hydrogen. How much of the carbon is free and how much is in combination with hydrogen will, most likely, never be known. The oxygen is, however, generally regarded as combined with the hydrogen, but it is not probable that this is the case with the deep dry coals containing occluded methane; some oxy-carbon compound containing hydrogen is probably present. The action of heat upon the coal will differ according as the hydrogen and oxygen are in direct union with each other or are in a position free to unite with each other and form water.

Parr⁴ reasons that volatile matter from coal is composed of two portions; the one composed of certain compounds of carbon

¹Econ. Geol., 1, 29; 1905.

²Chem. News, 70, 93.

³Phil. Mag. 352; 1882

⁴Ill. Geol. Surv. Bulletin No. 3; 1906.

and hydrogen which are combustible, and the other "a compound of hydrogen and oxygen in the proper ratio to form water, and hence, non-combustible". It is obvious then that if the hydrogen and oxygen are already in combination in the coal, the water thus formed simply increases the extraneous matter in the coal. Since the value of a fuel depends upon the amount of material in the same, capable of being oxidized, it is reasonable to say that this water serves no purpose. For economy's sake fuel should be as concentrated as possible, since there would be a saving in freight, as well as a more efficient combustion in the furnace. For two equal weights of the same oxidizable material, the one containing the less inert matter will tend to have the higher thermal efficiency.

Coalite.—During the year 1907 a good deal of notice was given in the British press to a product which is of interest in this connection. The following description is abstracted from a series of articles published in *The Iron and Coal Trades Review*, during 1907. The process for making coalite has been patented in the United States, England and Germany. The British patent claims that the method consists in subjecting any bituminous coal to a temperature approaching 800° F., (426° C.) in closed rectangular retorts, placed vertically in a gas-fired furnace for about eight hours, or until the illuminating gas ceases to be evolved. When illuminating gas ceases to come off, and the mass is substantially free from tarry components, the heat is suddenly arrested by the introduction of steam, and the product removed from the retort. Each retort has a capacity of 15 cwt.; and each charge yields about 11 cwt. of coalite. This may vary with different coals, but the yield will generally be about 70 per cent. According to the claims made for coalite, the yield and by-products will compare favorably with that obtained in the manufacture of illuminating gas.

The analysis of coalite is stated to be:

Ash.....	7 per cent
Volatile Matter.....	12 per cent
Fixed Carbon.....	80 per cent
Sulphur.....	1 per cent
B. t u.....	13500

The samples which have been on exhibition in London resemble coke in appearance and combustion, burning with a bluish

flame. The English press seems to be unfavorably disposed towards coalite, probably on account of the claims made by the promoters, that it is superior to any form of steam coal, and because of the diligent efforts to float the stock of the company. This company is capitalized at \$275,000 with permission to increase to \$10,000,000, thus giving it many of the attributes of a mere stock-jobbing enterprise. The total profit is estimated, in the prospectus, at \$2,500,000 a year from the production of 2,100,000 tons of coalite and the resulting gas, apart from other by-products.

The Scottish Smokeless Coal Company claim to make from non-coking smalls a fuel having an analysis practically the same as coalite. The Gas Light and Coke Company, of London advertise a smokeless fuel under the name of "carbo". The South Metropolitan Gas Company will probably place a similar product on the market. From these and other considerations, a serious question has been raised as to the validity of any patent intended to cover the process as outlined.

Mention is thus made of the main features involved in connection with "coalite" because of a certain resemblance in method to the experiments as carried on in this laboratory for the five or six years past. Our own work, however, has been directed primarily to the development of the fundamental facts and principles involved with a careful study of the chemical changes or reactions that may accompany the treatment under varying temperatures and in different atmospheres. The information resulting from such experiments is considered a necessary prerequisite to any industrial application of the process.

PRELIMINARY EXPERIMENTS ON THE MODIFICATION OF THE COMPOSITION OF COAL

During the coal famine of 1902, mainly centering in the anthracite regions, but affecting all parts of the United States to a greater or less extent, certain experiments were begun here in the chemical laboratory of the University of Illinois, suggested, or at least stimulated by the events of that year. These experiments had in mind the possible modification of bituminous coal in such a manner as to eliminate largely the constituents which tend to produce smoke in combustion and which would consequently give to the material the essential properties of anthracite or semi-anthracite

coal. The results were first published in the Year Book of the Illinois State Geological Survey for 1906, under the title, "The Anthracizing of Bituminous Coal,"¹ and when tabulated showed a possible increase in the fixed carbon of 25 per cent and over. These experiments, however, were of a preliminary type and took no account of the composition of the evolved gases nor of the varying effects that might be produced by different kinds of atmosphere. The fact that there was a decided increase in the percentage of fixed carbon furnished sufficient ground for interest and a desire to know the ultimate possibilities in this direction.

In continuing these experiments, therefore, the preliminary work already referred to indicated in a general way the type of apparatus needed and the conditions under which further tests should be made. It was evident that for a proper study of the changes taking place some tests must be made in a non-oxidizing atmosphere while others should introduce that condition in order to demonstrate the effect on the various reactions involved.

Mr. Deane Burns², in his thesis investigations, had made use of a small metal cylinder of six to eight grams capacity, which was brought under temperature control by being fitted into a hot air bath. With this device some attempt was made to govern the kind of atmosphere in which the distillation should proceed. In the tests herein recorded, a larger furnace with definite circulation of atmosphere was provided as in the description below.

EXPERIMENTAL WORK

FIRST SERIES: NITROGEN ATMOSPHERE

The first series of experiments was made in a non-oxidizing atmosphere in order to eliminate, as far as possible, those variables which would result from oxidation. While this would be an extreme condition, and one not possible as an industrial feature, for experimental purposes it would admit of the study of actual changes taking place as a result of heat alone. For this purpose, therefore, nitrogen, free from carbon dioxide and oxygen, was employed. This gas was prepared from air, as suggested by Hulett³. The apparatus and amount of material were slightly modified, in order to permit of easy observation of the process. As claimed by the author, it is not difficult by this method to pre-

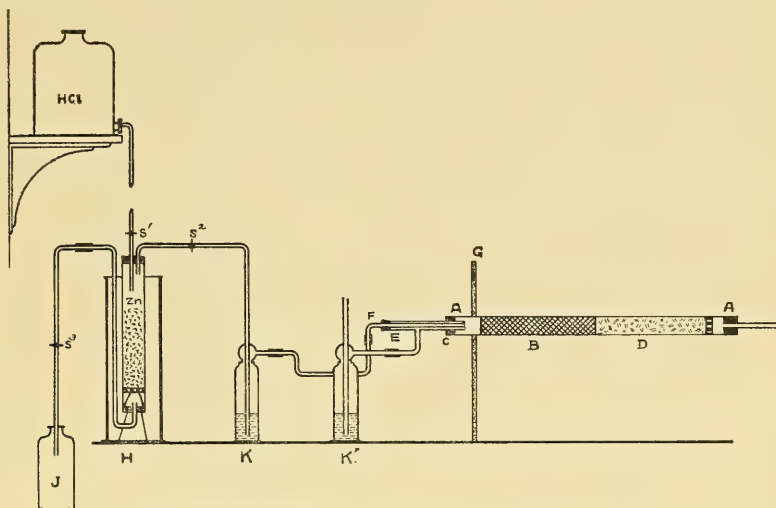
¹Ill. State Geol. Surv. Bulletin No. 4, Year Book, 1906. p. 196.

²Burns, Univ. of Ill. Thesis, Class 1907.

³Jour. Amer. Chem. Soc. 27, 1415; 1905.

pare large quantities of nitrogen, and at a rapid rate, when once the operation is under control.

Apparatus.—The essential parts of the apparatus are shown in Fig. 1.



APPARATUS FOR PREPARING NITROGEN
Fig. 1

A. A. is an ordinary combustion tube, 2 cm. or over in diameter and 1 meter long; B is 40 cm. of loosely rolled copper gauze; D is 35 cm. of copper oxide, the wire or gauze form preferred; E is made from a piece of hard glass tubing; F is a small porcelain tube or pipe stem (a Rose crucible stem was used in our apparatus); G is a thick piece of asbestos board for protecting the stopper C from the heat: the wash bottles K and K¹ are half filled with water and serve to indicate the speed with which the air and hydrogen are being admitted. The generator H, devised by Messrs. McClure and Barker¹, needs but little description. The reservoir for dilute hydrochloric acid is placed about 2 meters above the table, the flow being controlled by stop cock S¹; J serves as a reservoir for the spent acid which siphons over from the generator.

The nitrogen was used as soon as possible after preparation and when analyzed, was found to be free from hydrogen and car-

¹Chem. Eng. 6, 109; 1907.

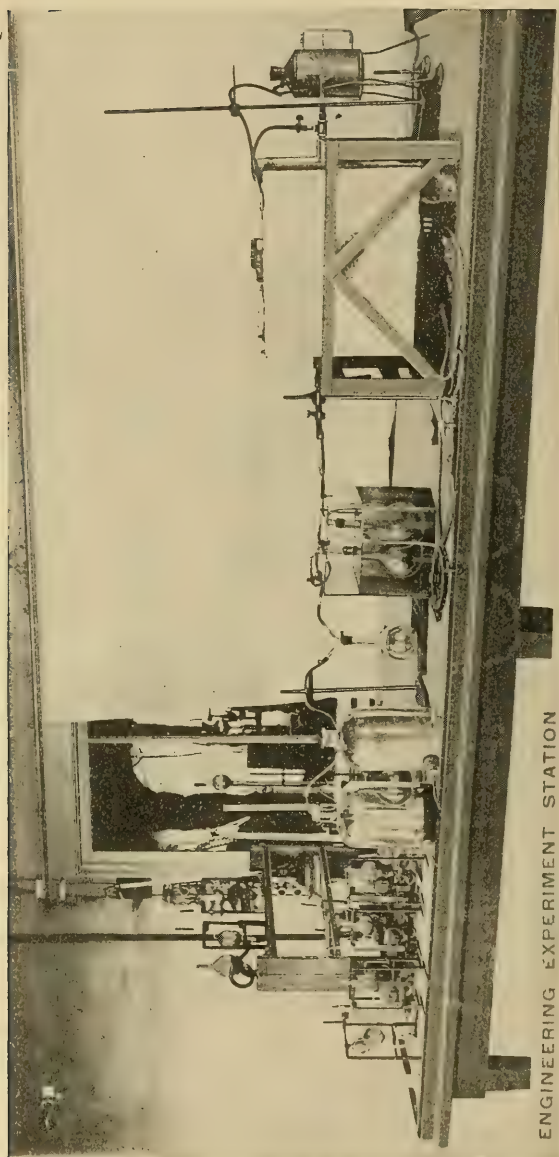
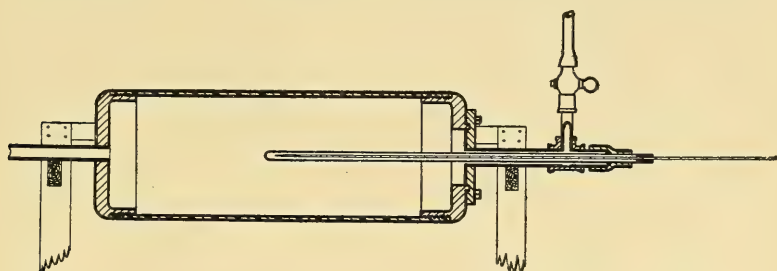


FIG. 2

bon dioxide, with but a very slight trace of oxygen, which may have been due to oxygen dissolved in the water of the gasometer.

The apparatus used for the experiments on coal is shown in Fig. 2. It consists of an iron retort placed in an oven, which is encased in asbestos. An exit tube leads into two flasks, kept cold with running water, which serve for collecting the liquid products of distillation. These flasks with their connections were weighed before each experiment; the increase in weight was called water and oil. The quantities of water and oil were determined later by separation. Beyond these is shown the safety flask, used to check any backward flow of water from the gas bottles into the condenser flasks. The details of the retort are shown in Fig. 3.



RETORT FOR LOW-TEMPERATURE
COAL DISTILLATION

FIG. 3

Temperatures were indicated by a mercury thermometer, with carbon dioxide under pressure above the mercury to admit of high temperature readings. This thermometer was protected by a glass tube, which was sealed into the retort by means of a piece of thick rubber tubing, being first pulled over it, and then over the end of the retort as shown in Fig. 3. This system permitted the thermometer to be withdrawn for readings without disturbing the atmosphere in the retort.

Material.—The bituminous coal used for this series of tests was a sample of No. 7 coal from Williamson Co., Illinois (Carterville), and was marked No. 686, for identification in this laboratory. It had the following composition:—

No. 686	Moist Coal	Dry Coal
Moisture	6.53	
Ash	7.76	8.30
Volatile Matter	33.86	36.23
Fixed Carbon	51.85	55.47
Sulphur	2.10	2.24
B. t. u. per lb.	12380	13244
B. t. u. Ash, Water, and Sulphur Free Basis		14567

The dry ash, 8.30 per cent, as shown in the above table, was used as a standard condition for reference. The results of the proximate analyses were calculated to the dry basis by subtracting the percentage of moisture from 100, dividing the remainder into each amount and multiplying by 100 to read as per cent.

$$\frac{51.85 \text{ per cent Fixed Carbon}}{(100 - 6.53)} \times 100 = 55.47 \text{ per cent Fixed Carbon, Dry Basis.}$$

The formula for calculating the B. t. u. in terms of "corrected" ash and water free, i. e., ash, water and sulphur free basis,¹ may be expressed as follows:

$$\frac{\text{B. t. u.} - (\text{Weight of Sulphur} \times 4050)}{100 - (\text{Ash} + \text{H}_2\text{O} + \frac{8}{3} \text{ S.})} \times 100 = \text{B. t. u. per lb. Unit Coal.}$$

Example.

$$\frac{12380 - (.0210 \times 4050)}{100 - (7.76 + 6.53 + (\frac{8}{3} \text{ S} \times 2.10))} \times 100 = 14567 \text{ B. t. u. per lb. Unit Coal.}^2$$

Operation, Mechanical.—In preparing the sample for this series, a large quantity, about 40 pounds, of the air-dried coal was crushed to buckwheat size, thoroughly mixed and divided into several portions, each lot being numbered to correspond with the test in which it was to be used.

The operation may be described as follows: The portion of coal, usually something over 2000 grams, was placed in the retort, Fig. 3.

The head of the retort was brought to place by the screws shown, and a perfect seal secured by means of asbestos packing

¹Jour. Amer. Chem. Soc. 28, 632; 1906.

²Trans. Amer. Inst. Min. Eng. Bulletin No. 19, 49; 1908.

³The expression "Unit Coal" is here used to denote the ash, water and sulphur free material.

moistened with water. The apparatus, when connected, was thoroughly tested for leaks. In order to wash out the air and to furnish an inert atmosphere, nitrogen was admitted until the gas at the exit tube would no longer support combustion; usually about 15 liters were required. Heat was then applied and the evolved gases collected in the gas holders, two of these always being attached, one cut off and held in reserve to be used in case there should be an extra amount of gas suddenly evolved or when the first was filled.

The retort was turned by hand every minute or two during the operation. The exit tube was polished and coated with powdered graphite in order to permit of its turning readily within the rubber tubing which leads to the condenser flasks. When the experiment was completed the retort was disconnected from the rest of the apparatus, then sealed with rubber stoppers and slowly cooled. The gas holders and the flasks were closed by means of the pinch cocks shown in the illustration, Fig. 2.

Operation, Analytical.—The gas holders were sealed when full, the time and temperature recorded and the bottle marked. The amount of gas was easily obtained as the capacity of each bottle had been determined. Each sample was carefully analyzed according to the methods devised by Hemple.¹

It may be advisable to mention that if the fuming sulphuric acid for the determination of illuminants shows a tendency to crystallize, slight warming, or better, dilution with water, will prevent this. Experiments seem to indicate that if this diluted acid will produce fumes and a hissing sound when added to water, it will absorb the heavy hydrocarbons. It may be of sufficient strength even when diluted with one-third its volume of water.

The coal before treatment and also the residue were analyzed according to the method recommended by the committee on coal analysis appointed by the American Chemical Society². The samples of coal as freshly taken, were air-dried for twenty-four hours, carefully sampled, then powdered sufficiently to pass a 60-mesh sieve. The method for moisture was slightly modified as follows: 1 gram was weighed into a small bottle, especially designed for the purpose³, provided with a ground glass stopper which fits over the outside edge of the bottle, thus preventing

¹Hemple's Gas Analysis, Trans. Dennis.

²Jour. Amer. Chem. Soc. 21, 1130; 1899.

³Ill. Geol. Surv. Bulletin No. 4, 195; 1907.

loss of material by contact with the ground glass surface when the dry coal is brushed out for the ash or other determination. The bottle and contents were placed in a toluene bath, the cover removed and the sample dried for 1 hour at 105° C., the cover replaced and put in a desiccator until cold.

The following table shows the conditions under which each test was conducted:

TABLE 1

TEST CONDITIONS: CARTERVILLE COAL; No. 686 and 791

Atmosphere of Nitrogen

Test No.	1	2	3	4	5	6	7	8
Weight of Coal in grams	2247	2278	2250	2465	2371	2400	2120	2500
Size of Coal	B.*	B.*	B.*	B.*	B.*	B.*	B.*	Nut
Period of Observation	2 hr.	2 hr. 20 m.	2 hr.	1 hr. 50 m.	4 hr.	6 hr.	3 hr.	6 hr.
Total time of Treatment	3 hr.	3 hr. 10 m.	3 hr.	2 hr. 40 m.	4 hr.	7 hr. 30 m.	5 hr. 20 m.	8 hr.
Lowest Temperature	214°	221°	204°	243°	208°	339°	369°	364°
Highest Temperature	286°	310°	299°	358°	316°	375°	431°	400°
Average Temperature	270°	292°	277°	336°	299°	367°	402°	381°

*Buckwheat.

The figures at the top of the columns indicate the number of the test. The period of observation is the time during which the heat was maintained, as nearly as possible, at the desired temperatures. The total time of treatment includes the period of observation and the time required to bring the apparatus up to the desired temperature. The highest and lowest temperatures indicate the extremes which were recorded during the period of observation.

In the tables below, No. 2*a* and 2*b*, the actual values of the resulting product are given, in comparison with values for the original coal, in order to show the entire alterations which have occurred.

TABLE 2a

PROXIMATE ANALYSIS OF THE RESIDUE COMPARED WITH THAT OF
THE ORIGINAL COAL. SAMPLE NO. 686
Atmosphere of Nitrogen

	Orig. Coal	1	2	3	4	5	6	7
Average Temperatures		270°	292°	277°	336°	299°	367°	402°
Moisture	6.53	1.79	1.01	1.53	1.44	1.00	0.70	1.25
Ash	7.76	8.71	8.85	8.85	8.96	8.48	9.20	10.85
Volatile Matter	33.86	33.93	33.58	34.52	30.37	34.13	25.34	19.95
Fixed Carbon	51.85	55.57	56.56	55.10	59.23	56.39	64.76	67.95
Sulphur	2.10	2.03	2.08	2.08	1.56	1.98	1.98	1.72
B. t. u.	12380	12946	13092	12980	13232	13282	13133	12836

TABLE 2b

PROXIMATE ANALYSIS OF THE RESIDUE COMPARED WITH THAT OF
THE ORIGINAL COAL. SAMPLE NO. 791
Atmosphere of Nitrogen

	Original Coal	Test No. 8
Average Temperature		381°
Moisture	6.38	0.44
Ash	8.92	10.40
Volatile Matter	34.64	23.73
Fixed Carbon	50.06	65.43
Sulphur	2.09	1.51
B. t. u.	12443	13192

In the above tables, No. 2a and 2b, the advantages exhibited in the product, in comparison with the original coal, are exaggerated by reason of the large loss of moisture. It is, therefore, misleading as an index of actual chemical transformations, but it does give, however, a correct relative indication of values in the treated condition. For example, the resulting product, in tests 4, 5, 6 and 8, shows a relative increase in B. t. u. of from 6 to 7 per cent.

In order to arrive at an appreciation of the actual changes that have taken place, the various constituents must be calculated to some common unit for comparison. This has been done in Tables 3a and 3b, where the unit employed has been the orig-

inal ash content of the coal reduced to the dry or water free basis.

TABLE 3a

RESULTING CONSTITUENTS COMPARED WITH UNIT ASH, DRY BASIS
 Carterville Coal. Sample No. 686
 Atmosphere of Nitrogen

	Values before Heating	Test Numbers						
		1	2	3	4	5	6	7
Average Temperature		270°	292°	277°	336°	299°	367°	402°
Ash	8.30	8.30	8.30	8.30	8.30	8.30	8.30	8.30
Volatile Matter	36.23	32.33	31.49	32.38	28.13	33.38	22.86	15.26
Fixed Carbon	55.47	52.95	53.05	51.68	54.87	55.05	58.43	51.98
Sulphur	2.24	1.92	1.95	1.93	1.45	1.94	1.89	1.32
B. t. u.	13244	12366	12278	12173	12257	13000	11847	9819
B. t. u. on Unit Coal* Basis	14567	14583	14639	14601	14859	14783	14688	14702

*By Unit Coal is here meant the ash and water free basis corrected for sulphur as on p. 18.

TABLE 3b

RESULTING CONSTITUENTS COMPARED WITH UNIT ASH, DRY BASIS
 CARTERVILLE COAL. SAMPLE NO.-791.
 Atmosphere of Nitrogen

Test No. 8	Values be- fore Heat- ing, Dry Coal	Values af- ter Heating, Referred to Unit Ash
Average Temperature		381°
Ash	9.52	9.52
Volatile Matter	37.00	21.70
Fixed Carbon	53.47	59.88
Sulphur	2.23	1.38
B. t. u.	13290	12076
B. t. u. on Unit Coal Basis	14819	14887

In these tables it is assumed that by taking a unit ash, for example, 8.30 per cent, or the amount present in the oven-dry coal before treatment, and calculating the relative amounts of the several constituents to this unit as a basis, an indication would thus be made of the actual changes produced in the several initial values. Thus, at all temperatures, there is an actual decrease

in the volatile matter which becomes marked in the higher temperatures, viz., in tests No. 4, 6, 7 and 8, or from a range of 330° to 400° C.¹

In tests 4 and 8, a positive increase of fixed carbon is shown, while in all cases a reduction in heat values is indicated. This reduction is accounted for by the hydrocarbon values represented in the gaseous and oil products of distillation. Especial attention should be directed to the last line, showing the heat values calculated to the unit coal basis. These values show a consistent increase throughout. A tentative explanation is offered in that the oxygen and nitrogen compounds of the volatile matter have been more largely driven off than was the case with the hydrocarbon compounds. If the loss in volatile matter, as shown, had been chiefly that of the marsh gas, (CH_4), series, a reduction in heat values for unit coal must result. If, however, the loss is made up of water of composition, there would be a relative increase in the heat content of the residual coal. The weight of water condensing in the flasks and separated from the oil, showed in each test an increase over the possible amount which could come from the free water present. The increase amounted to 3 per cent in test No. 4; 4½ per cent in test No. 6; and a little less than 3 per cent in test No. 7. These figures must represent the percentage of decrease in the water of composition. A loss of 2 per cent in this constituent would raise the B. t. u. factor, referred to the unit coal basis, from 14567 to 14864. This would seem to warrant the conclusion that a loss of water of composition occurs. This is an important point to further substantiate, as it is a fundamental feature of this investigation to develop, as nearly as may be, the conditions which govern the various decomposition processes.

In order to arrive at a further appreciation of the actual changes that have taken place, the alterations in the several constituents have been calculated to a percentage gain or loss of their original values as presented in Table 4, as below. From the table it will be seen that the actual loss in volatile matter has been accompanied in the higher temperatures, as shown in tests 6 and 8, with an actual increase of fixed carbon. The decrease in the heat units of the product is not an actual loss but is represented by corresponding values in the combustible gases as shown in a succeeding table, No. 5.

¹Centigrade degrees are used throughout in this discussion. $^{\circ}\text{C} \times 1.8 + 32 = ^{\circ}\text{F}.$

TABLE 4

LOSS OR GAIN OF CONSTITUENTS CALCULATED AS PERCENTAGE OF
ORIGINAL VALUES FOR EACH
Atmosphere of Nitrogen. Samples No. 686 and 791

No. of Test		Volatile Matter	Fixed Carbon	Sulphur	B. t. u.	Weight, Ash and Water Free	Yield per 100 lbs. Dry Coal	Average Temperature
1	Loss	10.75	4.56	14.28	6.63	7.00	93.58	270°C
2	Loss	13.08	4.38	12.94	7.29	7.81	92.84	292°
3	Loss	10.63	6.85	13.85	8.09	8.33	92.35	277°
4	Loss	22.35	1.08	35.27	7.45	9.49	91.30	336°
5	Loss	7.86	.76	13.39	1.84	3.57	96.73	299°
6	Loss	36.90		15.62	10.54	11.35	89.59	367°
	Gain		5.34					
7	Loss	57.88	6.30	41.11	25.86	26.68	75.54	402°
8	Loss	41.35		38.12	9.14	9.84	91.10	381°
	Gain		11.99					...

The character of the gas may be judged from the analyses given below, Table 5. No attempt was made to collect all the gas produced before Test No. 6. Although samples were taken and analyzed, they were not considered representative of the total volume, so are not given. The figures given below were obtained by averaging the analyses of each portion evolved.

TABLE 5

COMPOSITION OF GAS AS SHOWN BY AVERAGING ANALYSES OF PORTIONS
GIVEN OFF
Atmosphere of Nitrogen

Test No.	6	7	8
Temperature	339°C	369°	381°
Period of Observation	6 hr.	3 hr.	6 hr.
Carbon Dioxide and Hydrogen Sulphide	21.85	17.33	12.58
Illuminants	8.67	9.54	10.48
Oxygen	0	0	0
Carbon Monoxide	8.42	7.66	6.96
Methane	2.52	32.66	28.07
Hydrogen	1.99	2.37	2.07
Nitrogen	56.54	29.97	39.50
Volume of Gas Evolved	47 L.	50 L.	45 L.

EXPERIMENTAL WORK

SECOND SERIES: STEAM ATMOSPHERE

In this series, the atmosphere of nitrogen was replaced by one of steam. Presumably this also would be a non-oxidizing atmosphere, but the opportunity to study the action of steam directly, at the temperatures employed, as well as to compare the action with that where nitrogen was used, was deemed of sufficient importance to arrange for this series as given below.

The apparatus was set up as shown in the illustration, Fig. 2. The steam was generated from distilled water, and conducted directly into the retort, which was maintained, as before, at the desired temperature by means of gas burners, the retort being frequently turned on its axis. The coal treated was from the same mine as that used for the previous series. The composition of the samples is shown by the following analysis:

No. 1056	Moist Coal	Dry Coal
Moisture	3.28	
Ash	8.44	8.72
Volatile Matter	36.83	38.07
Fixed Carbon	51.45	53.19
Sulphur	2.49	2.57
B. t. u.	12868	13304
B. t. u., Unit Coal Basis		14605

The conditions under which this test was made are indicated below.

Coal No.	Test No.	Size of Coal	Weight of Coal	Period of Observation	Total Time of Treatment	Lowest Temperature	Highest Temperature	Average Temperature
1056	1	Buck-wheat	2400 g.	2 hr.	3 hr. 40 min.	366°	386°	381°

These results are very similar to those obtained in an atmosphere of nitrogen. It is to be noted that there is a relative increase in fixed carbon, (51.45 to 61.47) as also an actual increase (53.19 to 55.79). The relative heat values are higher after treat-

TABLE 6

CONSTITUENTS OF THE RESIDUE COMPARED WITH THOSE OF THE
ORIGINAL COAL

Atmosphere of Steam. Temperature 381° C.

	Proximate Analyses		Unit Ash Basis	
	Before	After	Before	After
Moisture	3.28	0.28		
Ash	8.44	9.64	8.72	8.72
Volatile Matter	36.83	28.51	38.07	25.78
Fixed Carbon	51.45	61.57	53.19	55.79
Sulphur	2.49	2.37	2.57	2.14
B. t. u.	12868	13221	13304	11959
B. t. u., "Unit Coal"			14605	14813

ment, but, when calculated to unit ash (8.72 per cent) the value is lower; the loss being represented by the hydrocarbons of the gases distilled. An interesting verification of the previous results is also shown in the B. t. u. calculated to unit coal. Here, again, it seems probable that the loss in volatile matter was greater in non-combustible constituents than in hydrocarbons. No approximation could be made in this test of the amount of water of composition recovered, because of the additional water due to the condensation of the steam. The additional weight of condensation would represent 8.30 per cent of water. In further experiments with a steam atmosphere, it may be possible to make a record of the amount of condensation from the steam introduced.

TABLE 7

ANALYSIS OF GAS EVOLVED FROM THE COAL UNDER AN ATMOSPHERE
OF STEAM

Coal No.	Temperature	Period of Observation	Hydrogen Sulphide	Carbon Dioxide	Oxygen	Illuminants	Carbon Monoxide	Methane	Hydrogen	Nitrogen	Volume of Gas
1056	381°	2 hr.	20.30	12.10	0.80	7.30	9.60	20.60	0	29.30	37 L.

EXPERIMENTAL WORK

THIRD SERIES: OXYGEN ATMOSPHERE

The operation was the same as in the previous series, except that an atmosphere of pure oxygen was supplied during the entire period of heating.

Material.—The coal treated was of the same character as No. 686, and from the same mine, with the composition shown in the following analysis.

No. 791	Moist Coal	Dry Coal
Moisture	6.13	
Ash	8.27	8.81
Volatile Matter	34.59	36.84
Fixed Carbon	51.01	54.21
Sulphur	2.06	2.19
B. t. u. per lb.	12443	13290
B. t. u., Unit Coal Basis		14819

The following table shows the conditions under which this series was conducted.

TABLE 8

TEST CONDITIONS. CARTERVILLE COAL. SAMPLE NO. 791

Atmosphere of Oxygen

Test No.	1	2	3	4
Weight of Coal	2350 g.	2200 g.	2000 g.	2350 g.
Size of Coal	Buckwheat	Buckwheat	Buckwheat	Buckwheat
Period of Observation	3hr. 10 min.	4 hr.	4 hr. 20 min.	4 hr.
Total time of Treatment	4hr. 20 min.	5hr. 15 min.	5 hr. 30 min.	6 hr.
Lowest Temperature	249°	328°	346°	349°
Highest Temperature	290°	358°	404°	402°
Average Temperature	279°	346°	379°	375°

The composition of the coal before and after treatment in an atmosphere of oxygen under the conditions as indicated, is shown in Table 9.

A feature to be noted in this series is the fact that, at a temperature of 279°C., only a small amount of decomposition has

TABLE 9
PROXIMATE ANALYSIS OF THE RESIDUE COMPARED WITH THE ORIGINAL
COAL. SAMPLE NO. 791
Atmosphere of Oxygen

Test No.	1		2		3		4	
	Be- fore	After	Be- fore	After	Be- fore	After	Be- fore	After
Average Temperature	279°		346°		379°		375°	
Moisture	6.13	0.82	5.99	0.82	5.97	0.75	5.03	0.46
Ash	8.27	8.61	8.26	8.90	8.01	9.67	8.10	9.68
Volatile Matter	34.59	35.06	36.01	30.80	35.19	21.25	34.01	25.27
Fixed Carbon	51.01	55.51	49.74	59.48	50.83	68.33	52.86	64.59
Sulphur	2.06	2.07	1.98	1.92	2.04	1.94	2.07	2.25
B. t. u.	12565	13027	12600	13251	12600	13152	12750	13217

taken place, or, in other words, the change in the constituents is about that which would result from the removal of the moisture. At 346°, as in test No. 2, a positive decomposition has occurred. It would seem, moreover, that oxidation had played a considerable part in the changes, as may be inferred by a reference to the composition of the gases from this test, shown in Table 11. The same statements may also be made in connection with tests No. 3 and 4.

TABLE 10
RESULTING CONSTITUENTS COMPARED WITH UNIT ASH, DRY BASIS
Atmosphere of Oxygen

Test No.	1		2		3		4	
	Be- fore	After	Be- fore	After	Be- fore	After	Be- fore	After
Average Temperature	279°		346°		379°		375°	
Ash	8.81	8.81	8.78	8.78	8.52	8.52	8.52	8.52
Volatile Matter	36.84	35.87	38.30	30.38	37.42	18.72	35.81	22.20
Fixed Carbon	54.21	56.80	52.90	58.69	54.05	60.20	55.66	56.85
Sulphur	2.19	2.00	2.10	1.88	2.16	1.71	2.17	2.98
B. t. u.	13385	13330	13403	13072	13399	11588	13425	11633
B. t. u., Unit Coal Basis	14819	14497	14813	14788	14768	14793	14800	14838

In this table as in tables No. 3a and 3b, ash is used as the basis of comparison in order to show the actual variation produced in the several factors. Since the samples were not identical, a slight variation in ash makes it necessary to compare the "before" and "after" values for each test. It is interesting to note that in an atmosphere of oxygen, the same general characteristics are evident as in Series 1 and 2. This additional fact is to be noted. An examination of the gas values as given in Table 11 shows a high percentage of CO₂ for tests No. 2, 3, and 4. This indicates a direct oxidation at the temperatures employed. The amount of oxidation would doubtless be in proportion to the volume of oxygen admitted.

TABLE 11

COMPOSITION OF GAS AS SHOWN BY AVERAGING ANALYSES OF PORTION GIVEN OFF

Atmosphere of Oxygen

Test No.	1	2	3	4
Average Temperature	279°	346°	379°	375°
Period of Observation	4 hr.	4 hr.	4½ hr.	3 hr.
Carbon Dioxide and Hydrogen Sulphide	5.25	20.80	12.73	13.84
Illuminants	0.00	2.50	3.53	4.24
Oxygen	13.80	10.40	9.27	7.68
Carbon Monoxide	2.50	6.60	4.74	6.64
Methane	7.20	12.27	13.68	15.16
Hydrogen	0	0	0	0
Nitrogen	71.25	47.43	56.05	52.44
Volume of Gas Evolved	12 L.	23 L.	50 L.	45 L.

The composition of the gas from one representative test in each series has been charted in Fig. 4, 5, and 6. These charts are intended to represent the analyses of different portions of the gas, the equal divisions being arbitrary. The temperatures given are those recorded when the receiver was opened for collecting the gas and when it was filled; or in other words, 109° to 184°. Fig. 4 indicates that the first portion analyzed was evolved during a range of temperature from 109° to 184°; the second portion from 184° to 375°, etc. In the case of nitrogen, (Fig. 4), and steam, (Fig. 5), the carbon dioxide shown to be present must be

looked upon as resulting from the residual oxygen in the gases or absorbed by the coal and thus being available when a sufficiently high temperature was reached.

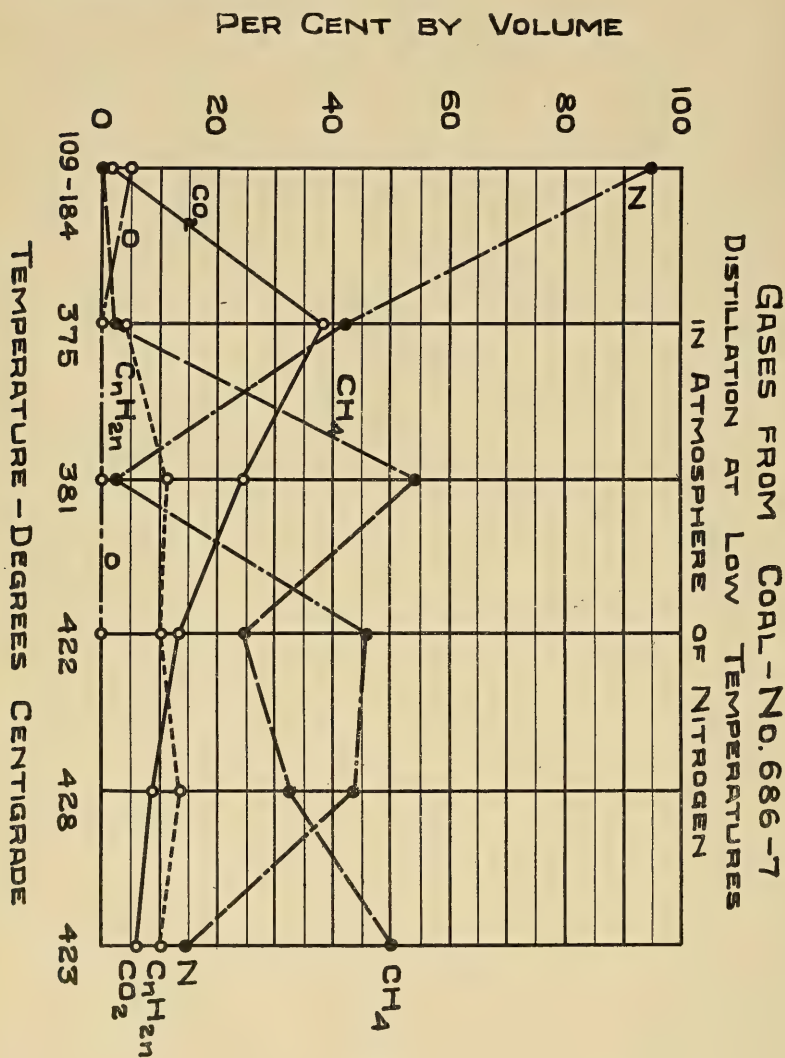


FIG. 4

EXPERIMENTAL WORK

FOURTH SERIES: OXIDATION EXPERIMENTS

A review of all the data up to this point makes evident certain conditions which call for specific investigation. For example, Tables 5, 7 and 11, and Fig. 1, 4, 5 and 6 show the composition of the gases given off in atmospheres respectively of nitrogen, steam and oxygen. In each case a very considerable content of carbon dioxide is present, ranging in amount from 12 to 27 per cent.

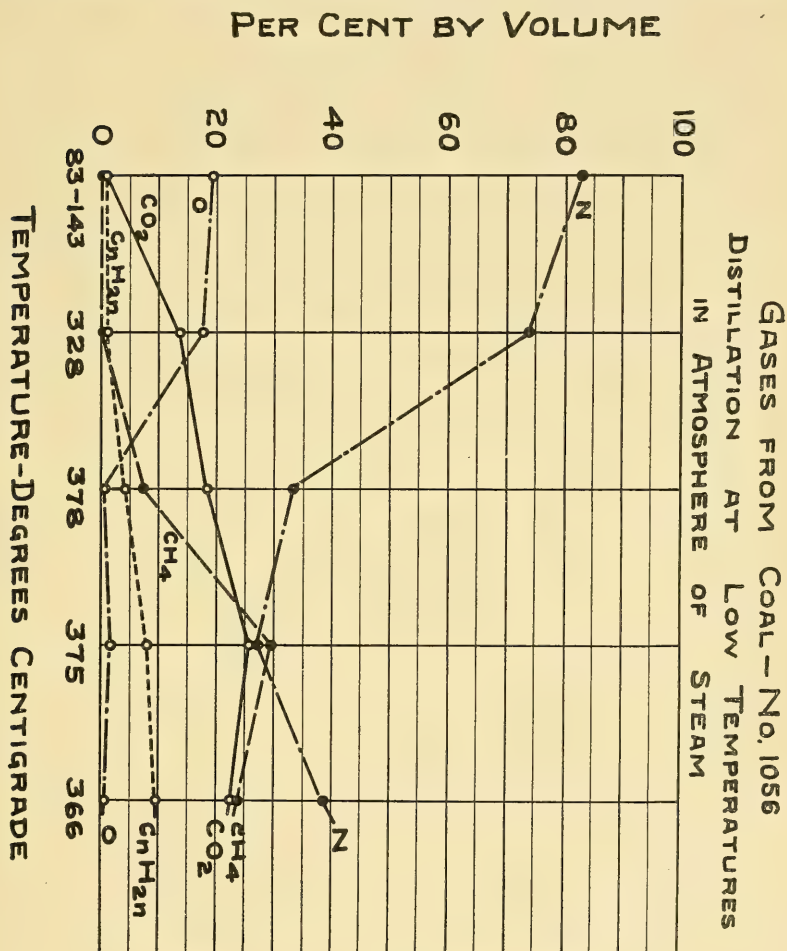


FIG. 5

Of related interest is the fact that in some of the tests with oxygen for the atmospheric medium, an occasional rise of the temperature in the retort was noted, seemingly independent of the external source of heat. This fact coupled with the marked amount of carbon dioxide in the gaseous product was positive evidence of active oxidation at the temperature of distillation. These facts

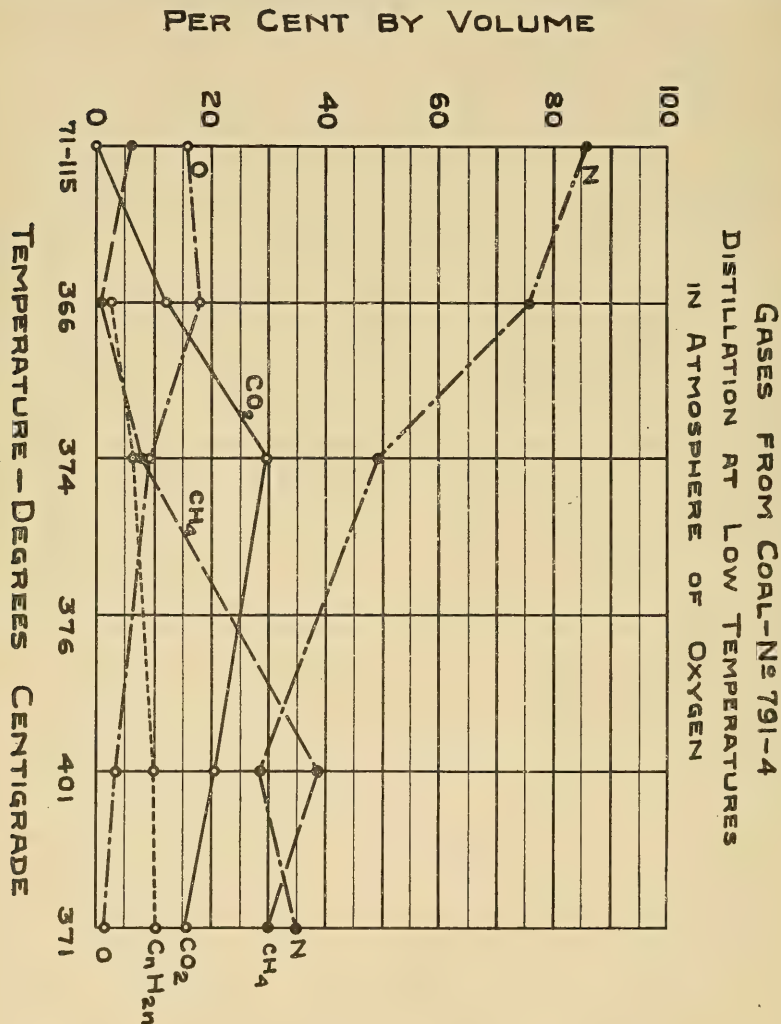
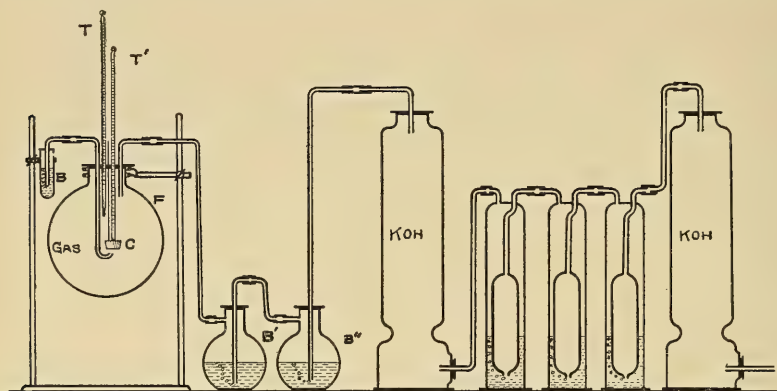


FIG. 6

also recalled certain phenomena in the earlier experiments conducted in a Rose crucible, with the thermometer bulb passing through the cover and immersed in the pulverized coal. At a certain temperature, considerably under 200° , a rise of the mercury seemed to occur, which was independent of the external source of heat. Among the tests described, where oxygen was the atmosphere used, test No. 4 showed this independent rise of temperature in a marked degree. In order to test further this evidence of oxidation, it was decided that at the end of an operation, the heat should be turned off and the retort allowed to cool slowly until a drop of, say, 50° had been recorded by the thermometer. At that point a fresh amount of oxygen was to be admitted and the temperature noted. Any rise of the thermometer would be proof of oxidation. After another drop of 50° , the test was to be repeated until a temperature was reached where no rise of the mercury would occur upon the readmission of oxygen. This would fix, broadly at least, the temperature at which active oxidation began. In following out this program, the end temperature of one test, (No. 4) was 375° . The source of heat was removed and the retort allowed to cool to 343° . Upon the readmission of oxygen copious fumes were shown at the exit tube. The thermometer was withdrawn far enough to make a reading which showed 349° . An obstruction seemingly hindered the free movement of the thermometer, necessitating its withdrawal, which was accompanied by a very marked additional rise of the mercury. The thermometer, moreover, could not be reintroduced, thus making necessary the dismantling of the apparatus. Upon opening the retort, the glass tube used to protect the thermometer was found to have been fused at a point just opposite to the oxygen inlet. By reference to Fig. 3, the relative positions of the inlet tube and thermometer may be seen. Fine particles of coal had sifted in about the protecting tube and furnished the material for the very active combustion at the point of inlet for the oxygen.

These facts suggested the desirability of a series of experiments wherein careful temperature observations could be taken with some means provided, also, for showing the presence of carbon dioxide, with a further possibility in the matter of atmospheric control which would permit the use of oxygen, steam, and air. These conditions were obtained in the following series of tests.

Apparatus.—The apparatus devised for the first tests with oxygen is shown in Fig. 7, and, as may be seen, consisted of two towers filled with solid potassium hydroxide, and three washing bottles partially filled with a 50 per cent potassium hydroxide solution. That this solution thoroughly removed any traces of carbon dioxide, which may have been contained in the oxygen, was proved by means of solutions of barium hydroxide in the two small flasks,



OXIDATION OF COAL
AND
TEMPERATURE MEASUREMENTS

FIG. 7

B' and *B''*. A round 1500 cc. Jena flask, *F*, served as a heating chamber; a nickel calorimeter capsule, *C*, for holding the material to be tested, was firmly fixed in a loop of heavy iron wire and suspended in the flask. Two thermometers were used, one, *T*, to indicate the temperature of the gas, (oxygen), and the other, *T'*, was immersed in the coal within the capsule. The exit tube led the products into a test tube, *B*, containing a freshly prepared solution of barium hydroxide.

Normally, it would be expected that the temperature of the surrounding gas would be slightly higher than that of the coal, the loss by convection and poor conductivity being shown by a slightly lower reading of the thermometer imbedded in the coal. It is evident, therefore, that any relative rise in temperature, as shown by the thermometer *T'*, would be due to chemical activity within the capsule. By charting the log of readings, therefore,

of these two thermometers, we have an index of the behavior of the coal. This plotting of the curves in the accompanying charts, therefore, shows at a glance the stages at which the changes occur, the crossing of the lines, or their relative directions, being due to the addition or removal of the exterior source of heat or to the greater or less activity of the oxidation process within the coal. This further point, however, should be borne in mind, that the temperature readings of the coal are relative as indicating the average value for the mass, since the oxidation, no doubt, is greater at the surface of the material, and the thermometer bulb must pass through zones of higher or lower temperature.

Operation.—The method of operation was as follows: Two grams of coal were placed in the nickel capsule and the thermometers, etc., adjusted as described. Oxygen was then admitted at the rate of approximately 150 bubbles per minute. The flask, *F*, was uniformly heated with a constantly moving Bunsen flame and readings of both thermometers were recorded every minute. The first appearance of carbon dioxide was noted in the test solution, *B*. This test tube was changed with sufficient frequency to indicate whether or not the evolution of carbon dioxide was continuous. It served to show also variations in quantity since it could easily be told by the rapidity of precipitation, whether the gas was increasing or diminishing in amount.

By referring now to the accompanying charts, the continuous line in each shows the reading for the surrounding gas while the dotted line gives the readings for the mass of the coal in the capsule. It may be said, also, that the readings were taken at half minute and minute intervals, but, for purposes of the charts, since the directions of the curves were not altered thereby, two minute intervals are indicated.

For convenience in charting, the results of any phenomena occurring between room temperature and those indicated, have not been employed. In most instances, however, there was a slight appearance of carbon dioxide at about 30° ; this disappeared until a temperature of about 125° was reached. From this it is inferred that a certain amount of carbon dioxide was occluded in the coal and was driven off at that temperature; therefore, the first appearance of that gas was not considered to be the point where active oxidation began.

The points to be noted are as follows: The crossing of the lines frequently occurs, showing that positive oxidation of the coal is taking place. If we examine in detail, for example, Fig. 8, which is for a sample of Carterville coal in a finely pulverized form, at the point indicated by the first cross, (+) or 125° , there was a positive appearance of carbon dioxide shown by the barium hydroxide solution. This appearance of carbon dioxide continued until a temperature of 155° was reached, when the chemical activity became so great as to cause a much more positive evolution of carbon dioxide and a very rapid rise of the thermometer T' ; at 168° , as indicated by the delta, Δ , the coal showed the presence of fire and, of course, thermometer observations could no longer be taken.

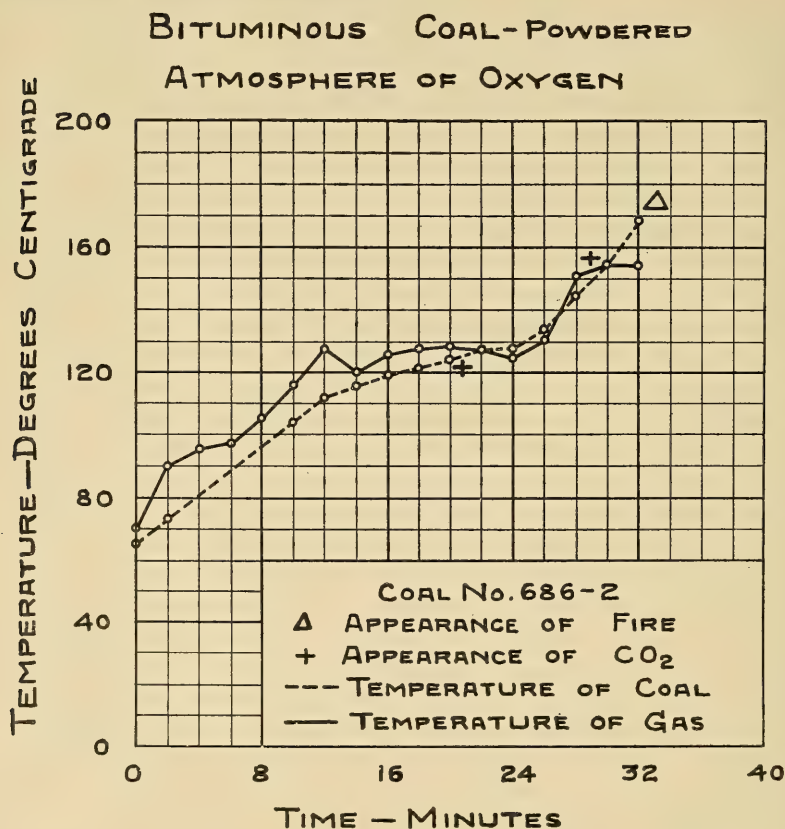


FIG. 8

Fig. 9 is a repetition of the previous tests as shown in Fig. 8, except that the coal was of buckwheat size instead of powdered. Carbon dioxide first appeared at an indicated temperature of 112° as shown by the first cross; at the second cross, which is intended to indicate the point where a very much more copious evolution of carbon dioxide appeared, the temperature reading was 147° .

BITUMINOUS COAL — BUCKWHEAT ATMOSPHERE OF OXYGEN

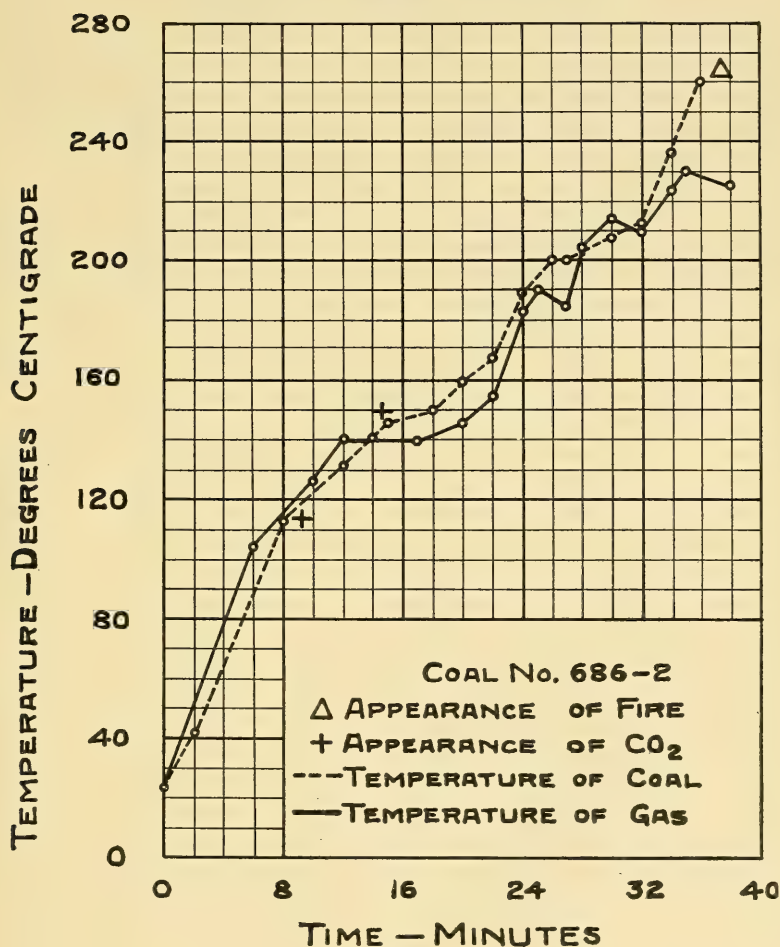


FIG. 9

This rapid evolution of carbon dioxide continued over a much longer space, however, and the activity was not sufficiently great

BITUMINOUS COAL — BUCKWHEAT ATMOSPHERE OF OXYGEN

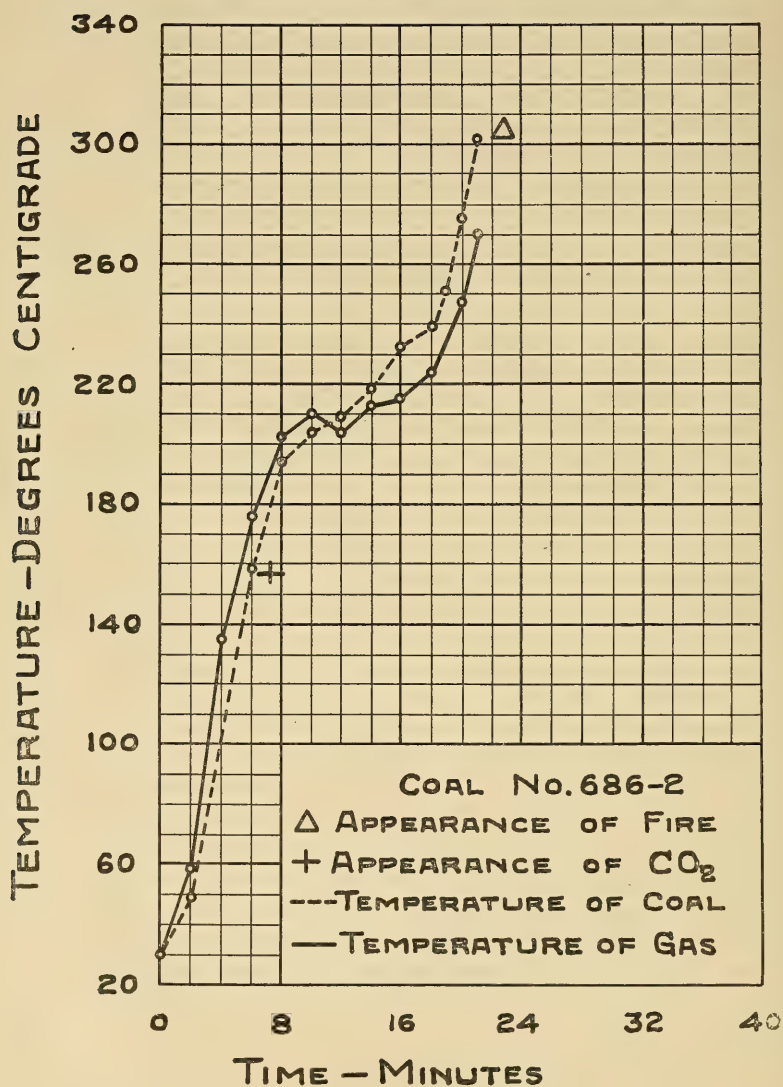


FIG. 10

to show a red glow within the coal until a temperature of 258° was reached. This simply shows that the oxidation could proceed upon the finely divided coal more rapidly than upon the buck-wheat size. A duplicate test on the same coal showed a decided oxidation at 138° , but the coal did not appear to be on fire until a temperature of 300° was reached; however, the general character of the curve for this test (see Fig. 9), corresponds with the original. In the first test the temperature of the coal was higher

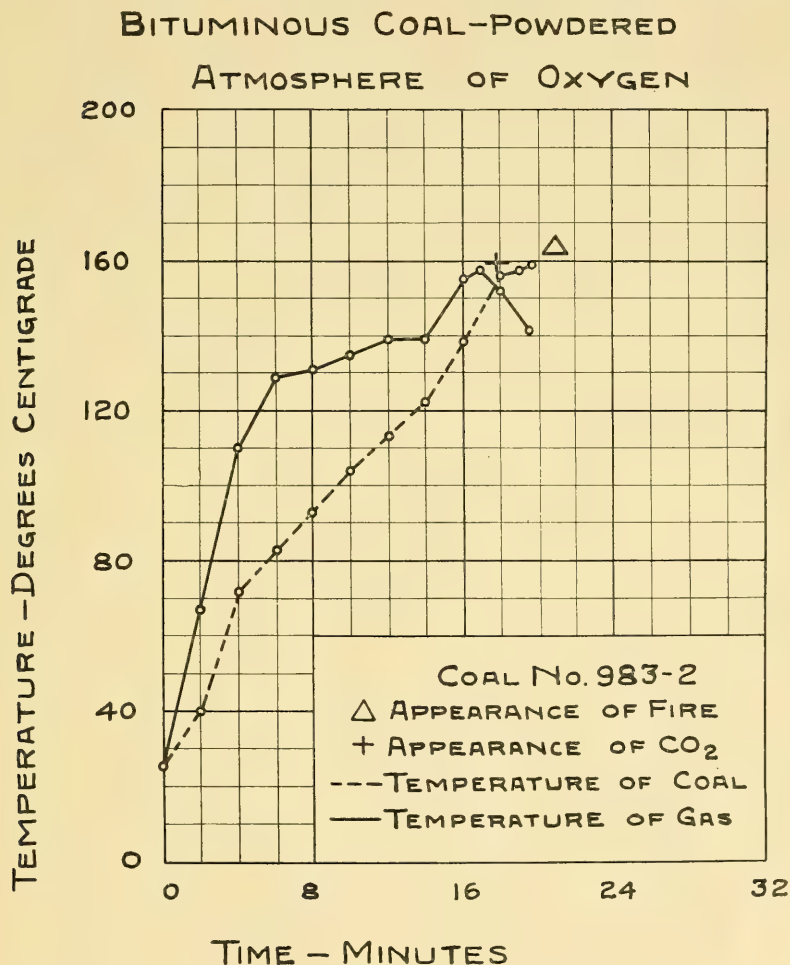


FIG. 11

than that of the gas after passing 140° , while in the second test this did not occur until 210° . This difference may have been due to the rapidity of heating, for, in the first experiment, the temperature was raised slowly, the entire time during which heat was applied being 34 minutes, while the second test was completed in 21 minutes. Under the latter conditions the oxidation was not so rapid as the increase from the external source.

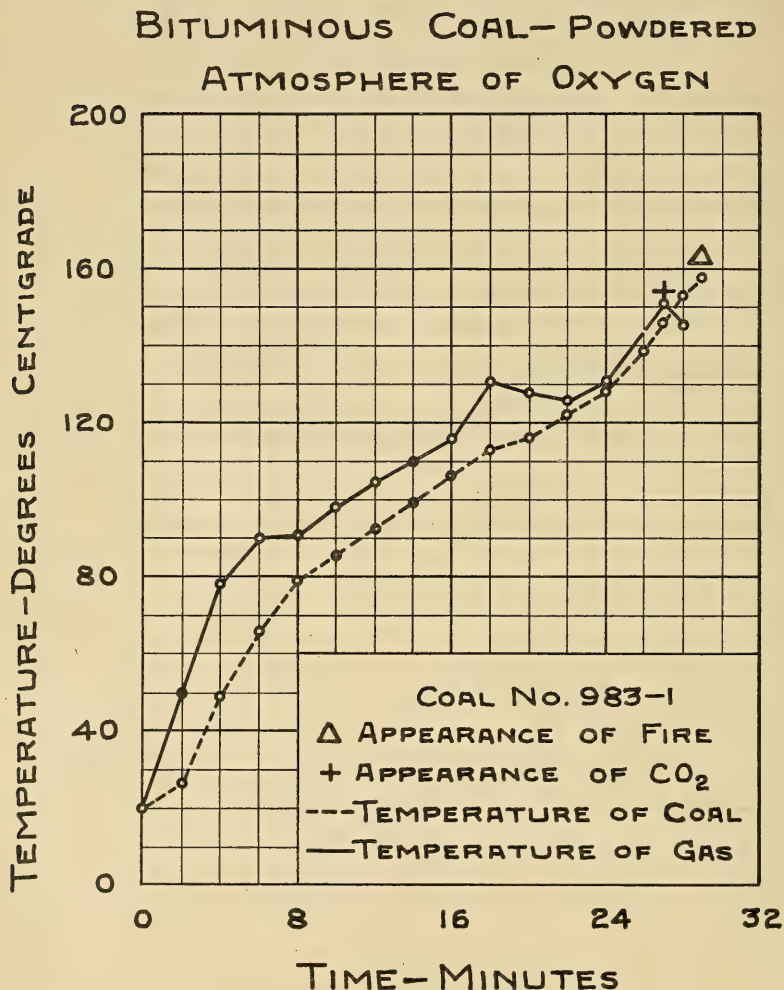


FIG. 12

Fig. 11, coal No. 983-1, represents an experiment on a sample of Taylorville, Illinois, coal in the powdered condition. The appearance of carbon dioxide was first detected at 153° C., and by reference to the chart it will be observed that at this point, the lines representing the two temperatures cross. Simultaneously, the evolution of CO_2 increased and at 157° , the coal was on fire. A second experiment on the same coal showed the point at which CO_2 was first evolved to be 156° , while the coal ignited at 158° (See Fig. 12, coal No. 983-2).

PITTSBURG GAS COAL-POWDERED ATMOSPHERE OF OXYGEN

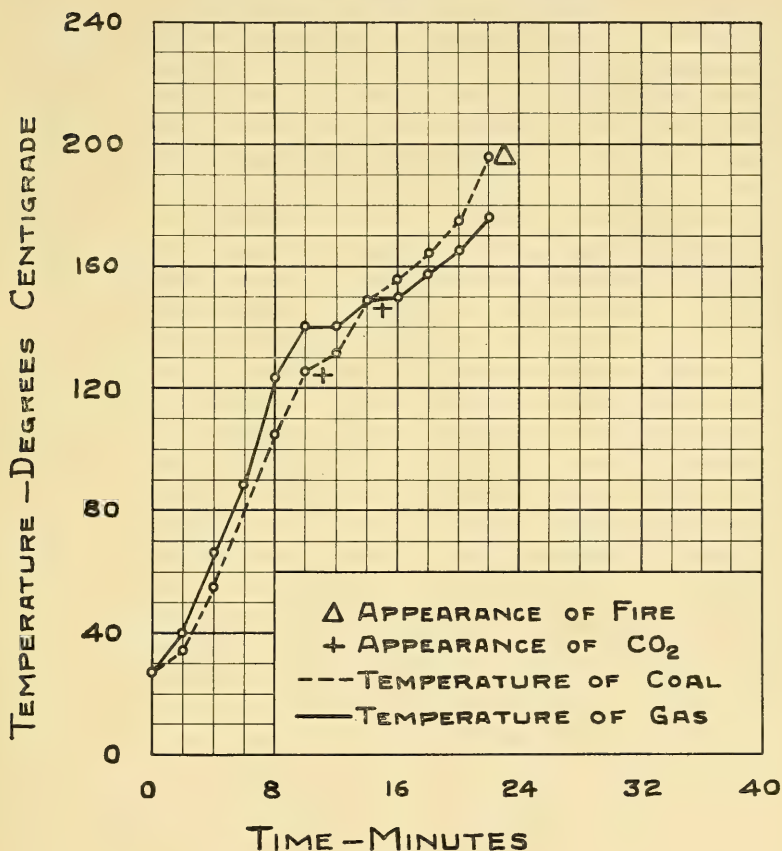


FIG. 13

Attention is called to the fact that no carbon dioxide was detected at the lower temperatures, 120° to 130°, as in the case of the other coals examined; also that the kindling temperature is practically the same as that at which the oxidation begins, as judged by the carbon dioxide produced.

ANTHRACITE — POWDERED

ATMOSPHERE OF OXYGEN

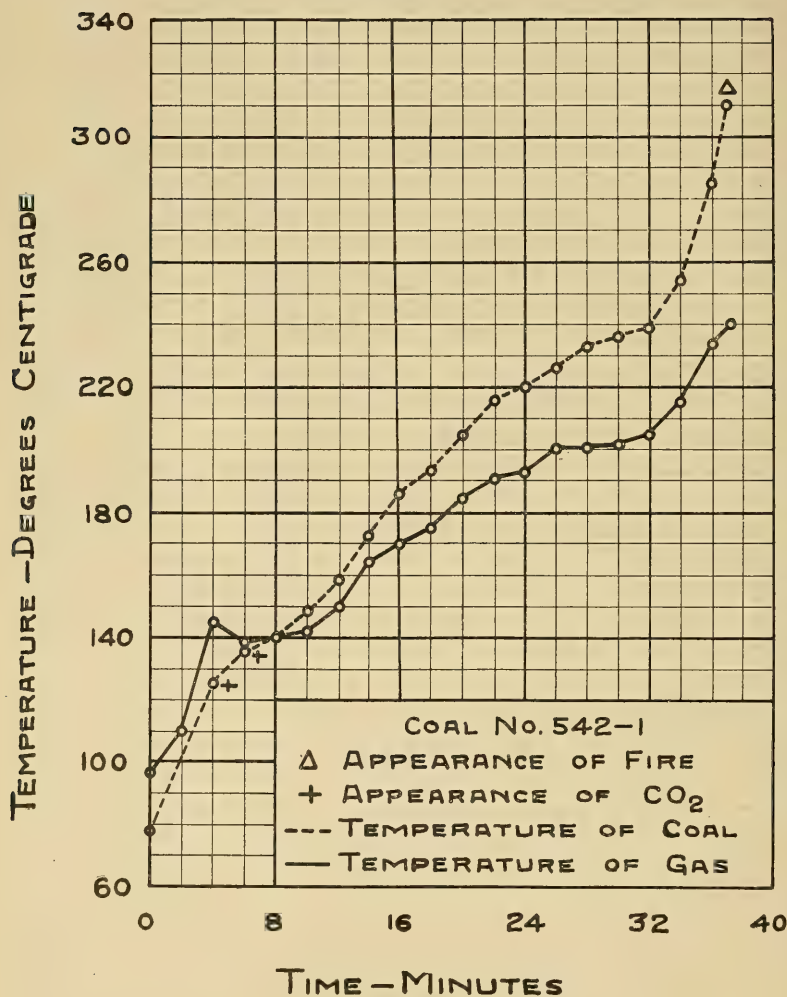


FIG. 14

In Fig. 13, a sample of Pittsburg gas coal in the powdered form, was employed. Here essentially the same phenomena were observed, as with coal 686-2 (Fig. 8), both as to the appearance of carbon dioxide and as to the more rapid evolution of the same, though the point for the appearance of fire was slightly higher than with the powdered bituminous coal of Fig. 8. This suggests that the oxidation of hydrogen may also have a part in the chemical reactions involved as being, perhaps, more readily available in coals of the strictly bituminous type.

In Fig. 14 the results are shown upon a sample of powdered anthracite. The first appearance of carbon dioxide was again at 125° . At 135° there is a stronger evolution of carbon dioxide, which continues with increasing rapidity as indicated by the more rapid rise in temperature up to 230° . At that point, a copious evolution of CO_2 occurred and ignition was indicated at 310° .

By reference to the literature of the subject, no very exact data for the kindling temperature of coal can be found. Lignite¹ is said to ignite at 150° ; anthracite, when in the powdered form, at 300° or above. Another writer² places the ignition temperatures of cannel coal at 370° C.; lignite at 450° , and Welsh steam coal at 477° .

More recently Moissan³, Manville⁴ and others have conducted experiments upon pure amorphous carbon to determine the variations in kindling point occasioned by oscillations of temperature and as modified by varying conditions of pressure, etc. There is some resemblance between the behavior of coal as here-in presented and the results by Manville upon amorphous carbon.

In order to correlate the facts of the Fourth Series with actual conditions such as would be met in the storage of coal, the atmosphere of oxygen there employed was replaced by atmospheric air. A number of tests were made using the apparatus of Fig. 7. One set of results, only, is produced, as in Fig. 15.

It will be seen that the results are not essentially different from those obtained in an atmosphere of oxygen. As would be expected, the dilution of the oxygen as in atmospheric air would retard or lessen the activity of oxidation so that the initial appearance of CO_2 was at 135° instead of at or about 120° , as in the

¹Ding. Poly. Jour. 64, 247; 12.

²V. B. Lewes, J. Gas Lighting, 55, 645.

³Compt. Rend. 135, 921.

⁴Ibid 142, 1190.

use of oxygen. The voluminous appearance of CO_2 and the crossing of the lines occur at 165° instead of from 140° to 150° , as with oxygen. Attention is also called to the fact that at about 200° the external source of heat being withheld for a few minutes, the oxidation temperature responded to that fact and dropped back in a similar manner, showing that in order to maintain oxidation at that temperature, an external source of heat was necessary. However, at 280° , the rapid rise in the oxidation tem-

BITUMINOUS COAL-POWDERED

ATMOSPHERE OF AIR

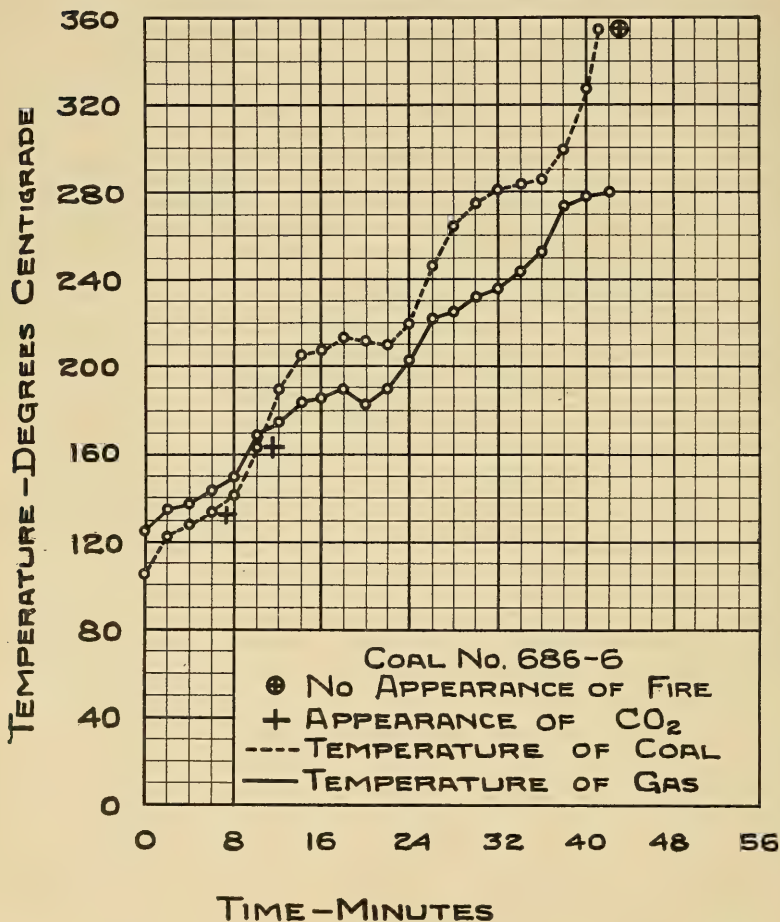


FIG. 15

perature continued, notwithstanding the cessation of the external heat. Seemingly, therefore, we have here an illustration of a type of combustion which, while still far below the ignition point, is still self-supporting and would be continuous, depending upon the oxygen supply. In another test under the same conditions essentially the same features were exhibited with the additional fact that at a temperature of 330° the coal ignited.

BITUMINOUS COAL-No. 686-7, POWDERED ATMOSPHERE OF STEAM

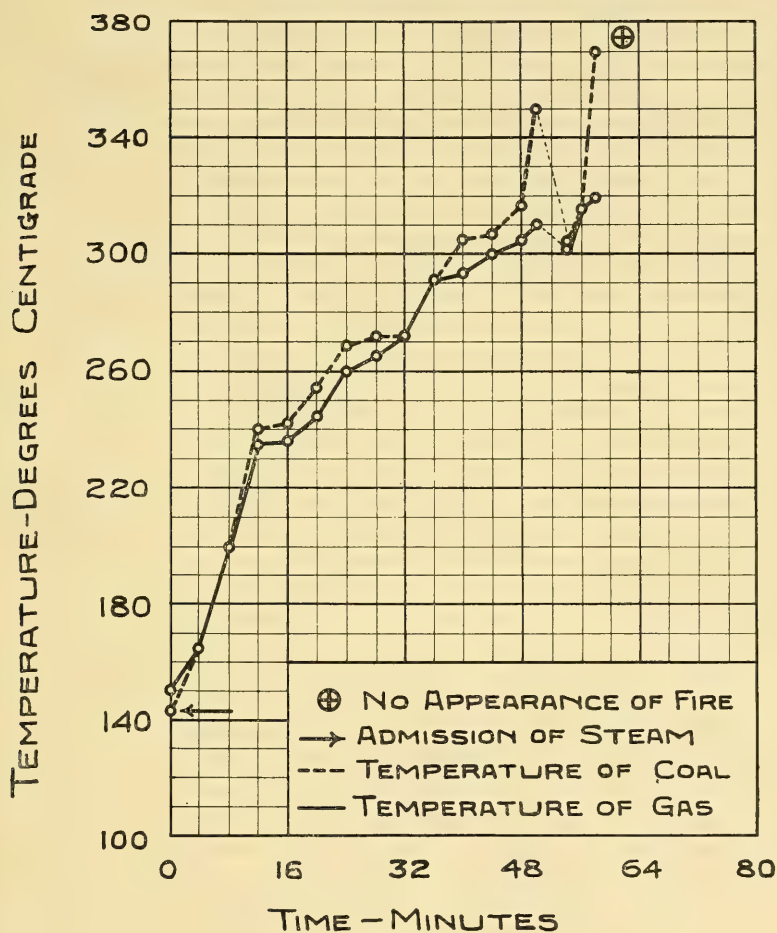


FIG. 16

EXPERIMENTAL WORK

FIFTH SERIES; ATMOSPHERE OF STEAM

By reference again to the composition of the gases from the retort as in Table 7 and Fig. 5, it will be seen that a very considerable amount of carbon dioxide is present. This may have been due to residual oxygen in the retort and, indeed, the decrease in quantity of the carbon dioxide as the process was continued (Fig. 5), would seem to indicate some such explanation for its presence. It was deemed necessary, however, to settle positively whether in an atmosphere of pure water-vapor any such formation of carbon dioxide could occur. This feature was investigated, therefore, by use of the apparatus already described, for studying the behavior in an atmosphere of oxygen, as illustrated by Fig. 7, the substitution of steam for oxygen without the purification train being the only change. No observations were taken below 140° , and, at about that temperature, the steam was admitted into the chamber *C*, Fig. 7. Duplicate tests were made on a sample of bituminous, (Williamson County) coal in powdered form and one test on the same coal in buckwheat form. The results of only one test are charted, as in Fig. 16.

An added precaution was taken in this test to have the bulb of thermometer *T* of Fig. 7 protected by a capsule exactly as the bulb of the thermometer *T'*, and the two were placed at the same level. Also, the inlet for the steam was carried down well below both capsules and discharged near the bottom of the chamber. By this means it was sought to eliminate as far as possible any external conditions which might cause a variation in temperature as between the two thermometers. The importance of taking this extra precaution will be evident from an examination of the curve in Fig. 16. A tendency is noted at a temperature of 240° for the thermometer embedded in the coal to show a higher reading than the other. This slight advance continued with more or less uniformity, but, at about the temperature of 315° , an altogether different phenomenon was noted. At this point the mercury rose abruptly, going almost at once to the limit of the thermometer used, while the other thermometer remained constant or showed only the slight rise due to the application of external heat. After noting this sudden rise in the first instance, the heat was removed and the temperatures allowed to drop to 300° . Upon renewing

the heat, a sudden rise was again observed when 315° was reached. No appearance of carbon dioxide accompanied this rise in temperature, hence, it must be attributed to the exothermic character of the decompositions occurring at that stage. The absence of carbon dioxide throughout the test limits us, for the present, at least, to this explanation of the temperature indications in the atmosphere of steam. Similar conditions were indicated in a corresponding experiment using nitrogen, but since a small residual amount of oxygen remained in the nitrogen, giving as a result a moderate test for carbon dioxide at the exit tube, this matter of temperature differences in nitrogen must await further and more careful examination. Indeed the general proposition here indicated, of a probably exothermic behavior, is of considerable importance and calls for a carefully devised series of experiments which are now in progress.

SUMMARY

The results thus far obtained are by no means complete but they are of sufficient interest to warrant publication in this preliminary form. Aside from the gases evolved, the by-products have received but little attention.

In the residual liquors, there seem to be no tarry compounds present but oils, phenols, ammonia, etc. Some quantitative results on the latter show as the highest result obtained from the wash liquors, about 11 per cent of the total nitrogen present in the coal. In most instances the yield was less than 1 per cent. This matter is receiving further attention.

Concerning the coal residue, enough has already been developed to indicate that it would have a special value for domestic use and such industrial operations as require a smokeless fuel. While much of the volatile constituent remains, it has undergone a change which makes it not difficult to carry on combustion without the production of smoke. This fact is, perhaps, suggested by the rather close resemblance in composition to the so-called smokeless coals. Because of the very great ease with which this material may be broken down, it would require, in all probability, to be subjected to the briquetting process.

As a rule, finely pulverized coals in contact with oxygen either diluted as in the case of air or in the pure state, begin oxidation at a temperature between 120° and 135° . In some in-

stances, however, this temperature of oxidation is higher, though in none of the tests did it exceed 155° .

The ignition temperature varies with the type of coal and to a certain extent also with the fineness of division. Finely divided bituminous coals ignite in oxygen at a temperature not far from 160° . Buckwheat sizes ignite at about 260° to 300° . Semi-bituminous coals ignite at about 200° and anthracites at about 300° .

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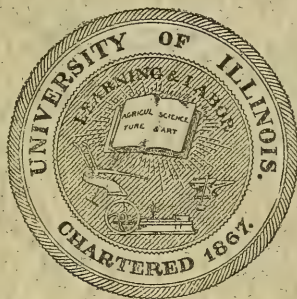
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BULLETIN NO. 25

LIGHTING COUNTRY HOMES BY PRIVATE ELECTRIC PLANTS

BY

T. H. AMRINE



UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

URBANA, ILLINOIS
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UNIVERSITY OF ILLINOIS

ENGINEERING EXPERIMENT STATION

BULLETIN No. 25

JULY 1908

LIGHTING COUNTRY HOMES BY PRIVATE ELECTRIC
PLANTS

BY T. H. AMRINE, FIRST ASSISTANT, DEPARTMENT OF ELECTRICAL
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STATION

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LIGHTING COUNTRY HOMES BY PRIVATE ELECTRIC PLANTS

INTRODUCTION

The farmer and the resident of the small country town have long felt the need of the electric lamp. They appreciate the adaptability, the cleanliness and the convenience of this method of illumination and would gladly adopt it in their homes, if possible. However, they live too far from any central lighting station to be able to buy power at a reasonable cost. The private lighting plant has been a possibility, but until recently the cost has been prohibitive for the great majority of people. The present state of development of the storage battery and the wonderful improvements that have been made in incandescent electric lamps during the past year have opened up to residents of the country new possibilities in the way of home lighting by private electric plants. The great difficulty in the design of a small lighting plant has always been the size and cost of a storage battery outfit. To start up the engine and dynamo every time a few lamps are required is too inconvenient to be considered. Consequently, it is necessary to have some means of storing the electric energy so that the power can be generated when convenient and used when required. The storage battery makes this possible, but where the ordinary carbon filament incandescent lamps are used as the light producing source, we are obliged to have a large and expensive battery. Each cell of storage battery will give an average pressure or voltage of 2 volts. Hence if 110 volt lamps are used, at least 55 cells of battery will be required. Moreover, when the battery is almost discharged, each cell will give only about 1.8 volts, so that if the lamps are burned at full brilliancy when the battery is almost discharged, a few extra cells will be required, making perhaps 60 in all. Since the carbon filament lamps are very inefficient light producers, the cells will have to be of large size, that is, have large lead plates and containing vessels, if they are to operate many lights for a length of time. Of course, the number of cells required may be decreased by using lower voltage lamps, say 25 or 30 volt lamps. The low voltage carbon lamps have an even lower efficiency as light producers than the 110 volt size. Hence, even though these lamps enable us to use fewer cells, each cell must be of larger size, if the lamps are to be burned as long as before without recharging the battery.

During the past year there has been put on the market a new type of lamp having a filament made of the rare metal, tungsten. With these lamps a given amount of energy will produce about three times the candle power that would be produced by an ordinary carbon filament lamp. Electrical energy is measured in watts, one of which represents $\frac{1}{746}$ part of a horse-power. With the 110 volt, 16 candle power carbon lamp it takes from 3.1 to 3.5 watts of energy to produce a candle power of light. With the tungsten lamp of the same voltage it requires about $1\frac{1}{4}$ watts of energy to produce a candle power. Consequently, when using the tungsten lamp for a private plant, it will take much smaller storage cells to operate a given number of candle power of lights than it would if carbon lamps were used. These new lamps are made, and are on the market, in 25 to 30 volt sizes. They give 20 candle power and consequently require only about 25 watts of energy to operate them. These lamps will make possible the cheapest kind of a plant.

Besides being cheaper to operate than the carbon lamp, these tungsten lamps will burn a greater number of hours. The carbon lamp will, under good conditions, burn about 500 hours, and at the end of that time, the candle power will have fallen 20 per cent; i. e., if when new it gives 16 candle power, at the end of 500 hours of service it will give but 12.8 candle power. The tungsten lamp, however, will often burn for 1000 hours, and during that time will lose only 10 per cent of its original candle power.

It is one purpose of this bulletin to trace through the design of a private electric plant of sufficient size to light a country home properly with 25 volt tungsten lamps. An endeavor will be made to show the steps in the design in such a clear and simple manner that with the information which can be obtained from the companies furnishing the batteries, engine and dynamo, anyone can decide upon the size of the equipment needed and order it intelligently. Full instructions for installing the apparatus and wiring are omitted, since it will be better to secure an experienced man to do this work. There will be given also in this paper an example of a lighting scheme for a medium sized private residence in which attention is paid to the proper selection and location of fixtures and shades.

CONCERNING ARTIFICIAL ILLUMINATION

It is a fundamental principle of good artificial illumination to keep the illumination of objects as strong as is required for the uses to which they are put and to keep the intensity or brilliancy of the lights as low as possible. The first part of this principle can perhaps be readily appreciated by the average person, but the second part is directly opposed to his conception of how lighting ought to be done. It seems to him that to get good illumination a great brilliancy is required, and that anything that reduces the brilliancy of the light source tends to decrease the quality of the illumination. To understand this part of the principle it must be remembered first, that intensity or brilliancy of a light source, for example, an incandescent electric lamp, refers simply to the amount of light coming from each square inch of surface on the light-giving source, that is, the filament. If a diffusing globe is put about the lamp the filament itself is not seen and the light will appear to radiate from the entire surface of the globe. With a properly made globe the amount of light that is lost in passing through the glass is small so that the total amount of light given off will be almost the same as from the bare lamp. The amount of light per square inch of the surface, that is, the intensity, is much less than before since it now radiates from the entire surface of the globe instead of from the small filament. It must also be understood how the human eye acts under lights of different intensities. The eye, by means of an adjustable opening, called a pupil, endeavors to receive always a constant amount of light by contracting or dilating as the light is intense or dim. When the light reflected to the eye from any object is intense the pupil contracts so as to shut out a large part of the rays. When light of only low intensity reaches the eye from any body, the pupil opens wide so as to admit sufficient light to enable the eye to see the object distinctly.

Imagine a room illuminated by an unshaded 32 candle power lamp hung rather low, and that we wish to see clearly a book on a table near the lamp. To see the book, of course, some of the light must be reflected from it to the eye. Since it is close to the lamp the book receives considerable light and it would naturally be supposed that sufficient light from it would reach the eye to enable us to see it clearly. So it would if the eye were free to

adjust the opening of the pupil to the intensity of the light that is received from the book. However, since the low hanging lamp itself is almost in the direct line of vision the rays from it are also reaching the eye. These rays are so intense that the eye to protect itself must almost close the pupil. In doing so it also prevents sufficient light from the book from reaching the interior of the eye, so instead of seeing the book clearly we see it only indistinctly and at the same time have an unpleasant or even painful feeling caused by the forcible contraction of the pupil. Because we do not see the book comfortably we are erroneously led to assume that the light is insufficient.

Suppose we place over the lamp a diffusing globe, for instance, a round frosted globe. The intensity of the light is now cut down a great deal, but the total amount of light is not greatly decreased. Now when we attempt to see the book the rays of light which reach the eye from the lamp itself are much less intense than before. Hence the pupil is left more widely open, and even though less light is reaching the book than when the lamp was unshaded, the eye is enabled to receive more reflected light from it, and the book can be seen more clearly. Moreover, because the pupil is not so closely contracted, the eye feels much more comfortable, and the dazzling effect is much decreased.

Let us make one more change. Let us raise the lamp high enough so that the direct rays from it will not reach our eyes when we look at the book. Now as we have taken the lamp further from the book so that it receives less light than before, we will remove the round globe and replace it with a tulip or bell-shaped shade. This will deflect the light from the lamp downward so that the book will receive about the same amount of light as formerly. Now when we look at the book, there is no direct light from the lamp reaching the eye. Hence, the pupil can adjust itself to receive the proper amount of light from the book, and, since the book itself is receiving sufficient light from the lamp, the eye will receive enough reflected rays from it so that it can be seen clearly.

In our attempt to illuminate the book so that it could be seen clearly and comfortably, it will be noticed that our efforts have been directed, first, towards getting the light upon the book and second, towards diffusing the light, or towards keeping the light screened from the direct line of ordinary vision. These results

should be the end toward which all efforts in illumination are directed. They are obtained by the careful placing of the lights, and by the use of proper shades and globes.

Contrary to the popular idea, the selection of shades and globes should not be made primarily with regard to their decorative qualities. Properly designed and constructed shades and globes are made either to send the light in some desired direction, to diffuse the light, i. e., decrease its intensity, or to combine the two purposes. A person selecting a shade for a light should then bear in mind the location of the light, where the strongest illumination is desired, and whether the light needs to be diffused. A shade or globe should then be selected that will fulfill the required conditions. Many manufacturers will furnish diagrams showing how each particular shade or globe made by them diffuses and distributes the light. From these diagrams the proper selection can best be made.

Unquestionably the best shades and globes are those made from clear transparent glass similar to the Holophane globes. These have the inner surface of the glass given over to the flutings or prisms used solely for diffusing and softening the light. On the outer surface there are flutings calculated to deflect these diffused rays into directions where needed. Although the material is clear, transparent glass, the prisms and flutings diffuse and reflect the light perfectly while at the same time there is but small loss by absorption. These shades are designed in three classes according to the service that is required of them. One class (A) throws the strongest illumination directly downward, the second (B) gives a strong illumination in all directions below the horizontal, while the third (C) throws the strongest illumination slightly below the horizontal.

Opal, opaline and ground glass globes and shades give a well diffused light, but there is a considerable loss of light by absorption. The ground glass globes have the disadvantage of being difficult to keep clean. If properly shaped, these globes will throw the light in almost any desired direction.

The ordinary plain glass shades having fancy designs etched upon them, such as are supplied with many electric light fixtures, are of little value except for what little decorative qualities they may possess. They change the distribution of the light to only a slight extent and the amount of diffusion is almost negligible.

Opaque metal and silvered glass reflectors are very satisfactory for deflecting the light in any desired direction, but they give no diffusion and always make a room look dark and cold on account of furnishing no light to the ceiling. They also give too great contrasts between intense light and darkness so that the pupil of the eye, as one looks from place to place about the room, must continually contract and dilate so that it is soon fatigued.

SELECTION OF FIXTURES AND PLANNING OF LIGHTING ARRANGEMENT

Since the sole object of an electric light plant is to provide illumination for the house, it is common sense to plan a good lighting scheme and then build a plant and install wiring in accordance with this scheme. This statement is called forth by the fact that the opposite course is usually pursued. The wiring is usually installed and the outlets for the lighting fixtures placed in a sort of a haphazard way at any convenient spot. Ordinarily they are placed directly in the middle of the ceiling whether or not that is the position most desirable from the standpoint of proper illumination of the room.

We will assume as a house for which we are going to design an electric lighting system, a country home having, on the first floor, a living room, a dining room, a kitchen, a front and a rear hall, a bed room and a large porch in front. On the second floor there are four bed-rooms, each provided with a closet, a bath room and a hall. In the cellar there is a large furnace room, a fuel room, a laundry, a vegetable room and a store room. Plans of the two floors and the basement are shown in Fig. (1a), (1b) and (1c).

Living Room.—Since this is the room in which most of the leisure time of the family is spent, it should be well lighted. First of all there must be a light for reading purposes. Since the family is likely to be large, several persons will often want to read at the same time, so a considerable area should be well enough illuminated to enable one to read anywhere within that area. When looking about for a lamp to furnish a light by which to read, the average person promptly selects a table reading lamp. The ordinary table electric reading lamp would be very satisfactory for one or possibly two persons to read by if a general illumination for the room were taken care of by other lamps.

In the ordinary farm home, however, usually more than one or two persons wish to read at the same time. Moreover, the lamp that furnishes light for reading is usually required to furnish a general illumination for the room. This a table lamp will not do. Accordingly, a three light fixture is provided as shown in Fig. 2.

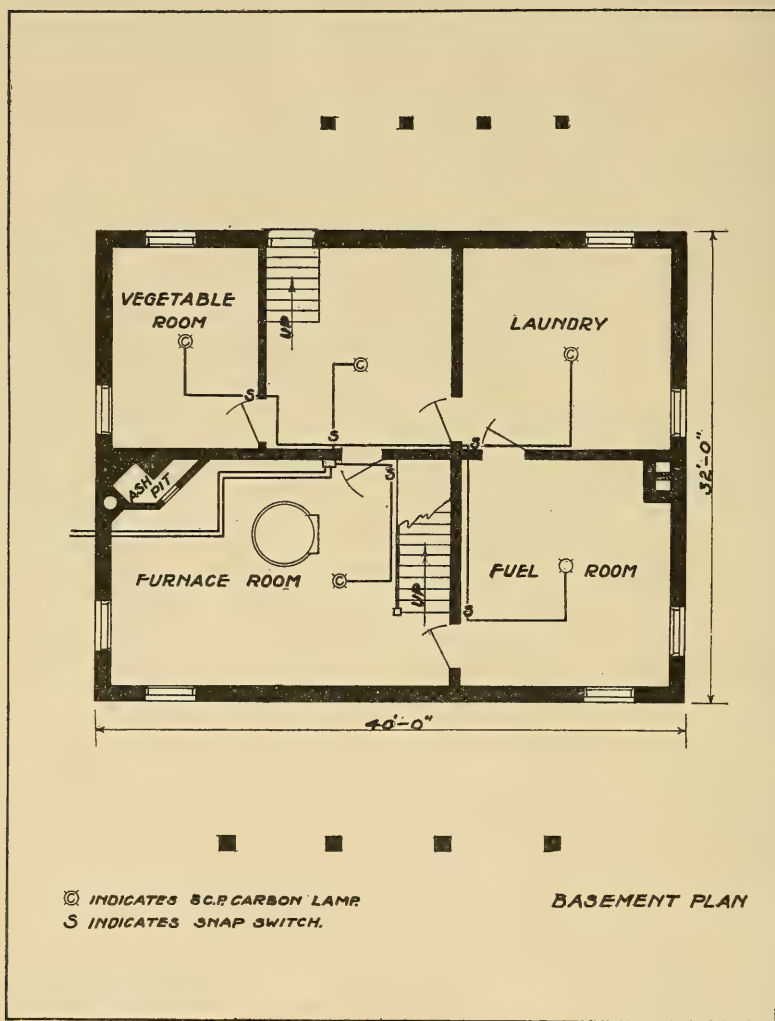
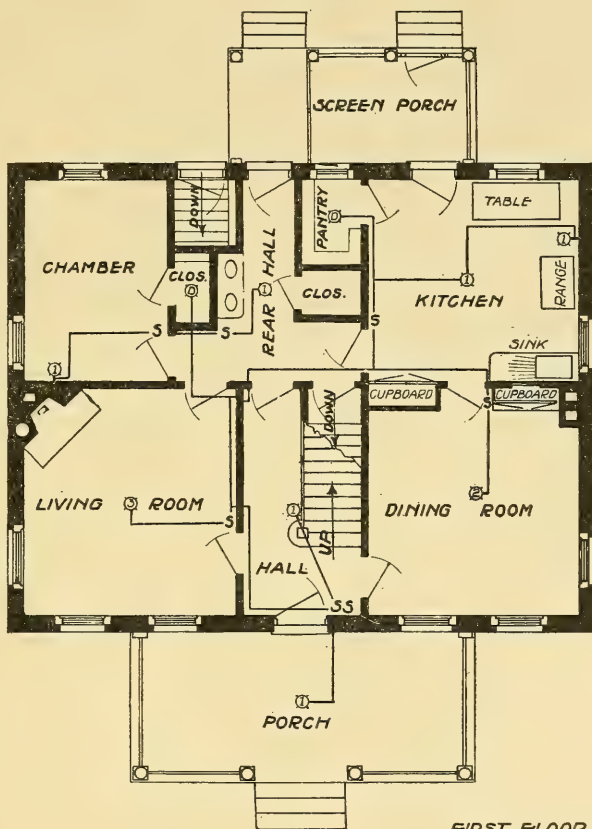


FIG. 1a



FIRST FLOOR PLAN

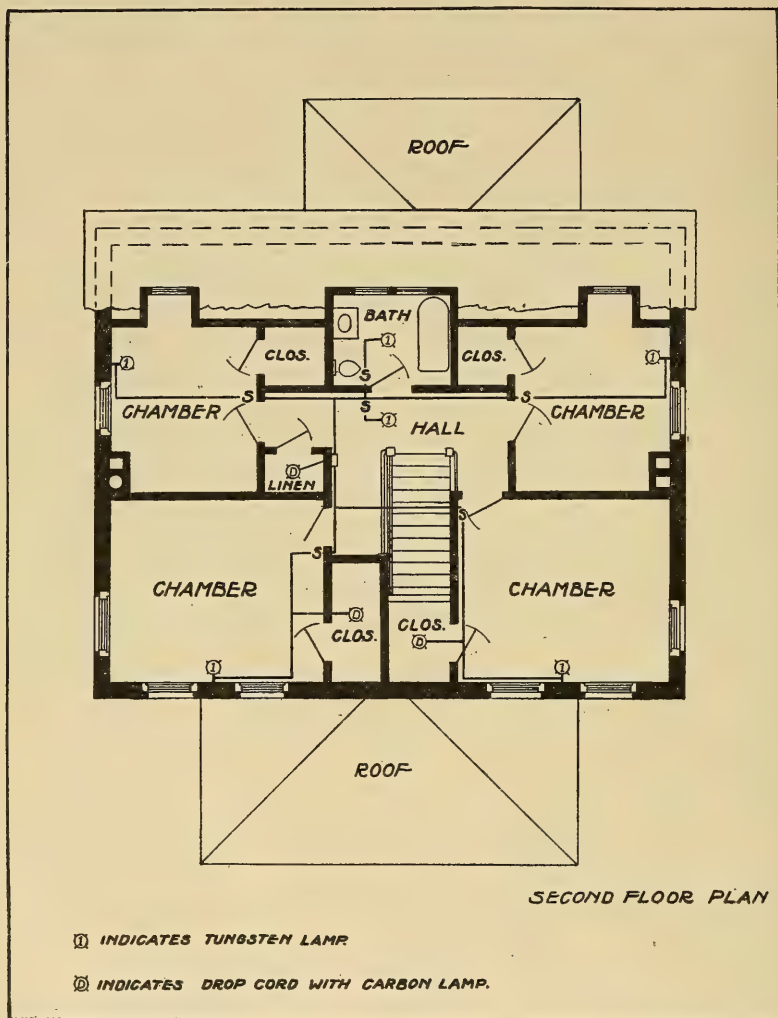
② INDICATES NUMBER OF TUNGSTEN LAMPS.

⊙ INDICATES DROP CORD WITH B.C.R. CARBON LAMP

S INDICATES SNAP SWITCH.

ENGINEERING EXPERIMENT STATION

FIG. 1b



ENGINEERING EXPERIMENT STATION

FIG. 1c

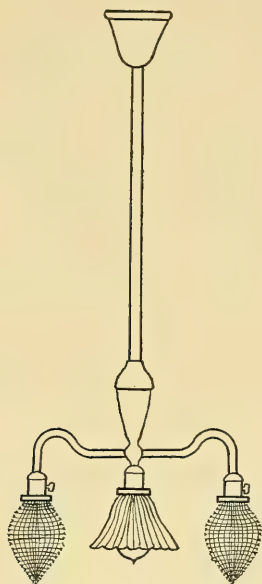


FIG. 2 LIVING ROOM FIXTURE

In this fixture the middle socket points directly downward, and is equipped with a prismatic glass reflector such as the Holophane, class B, "tulip" reflector. This will concentrate the light under the chandelier for reading purposes and at the same time permit sufficient light to pass through to give a moderate illumination of the walls and ceiling. Thus, the single reading lamp would be sufficient for ordinary occasions. On special occasions, however, when a general illumination rather than a concentrated light for reading is desired, the middle lamp is turned out and the two outside lamps are used. These two lamps are provided with prismatic reflecting globes of the shape shown. (Holophane, class B, stalactite). Since the reflecting globe will prevent the dazzling direct rays from the filament from reaching the eyes of a person in the room, the unfrosted lamps may be used. The middle lamp, however, may be seen from positions close under the chandelier. Hence a frosted lamp should be used here. The fixture should be hung so that the lamps are about six and one half feet from the floor.

Dining Room.—A dining room requires a strong illumination over the table and a soft pleasing light over the walls and

ceiling. This can be obtained for the room we are considering by two lamps placed in prismatic bowl reflectors hung at a height of six feet from the floor. These reflectors will distribute the light well to the edges of the table, while the ceiling and walls will be sufficiently lighted to make the room seem cheerful, but not brilliant. Frosted tip lamps should be used.

Front Hall.—A single unfrosted lamp placed in a Holophane, class B, stalactite, similar to the one shown in Fig. 2 will amply light the hall. It should be hung about eight feet from the floor so as to throw the strongest illumination toward the door and the foot of the stairs.

Kitchen.—The kitchen holds such an important place in the life of the farm house-wife that it should be well illuminated. It can be adequately lighted by a single lamp in a pendant fixture in the middle of the room. This should be provided with an opal bell reflector. The fixture is hung high so as to be out of the direct line of vision of a person in the room. To provide a more concentrated light over the stove and table where it is most needed there is an adjustable bracket fixture with an opal bell reflector and a frosted tip lamp. (Fig. 3.) This should be placed about six feet high and as nearly as possible between the stove and table. The lamp will be turned on only when needed and the light directed where desired by adjusting the bracket.

Front Porch.—One lamp placed inside of a prismatic reflecting ball similar to the one shown in Fig. 4 is used for lighting the porch. This is placed in front of the door and directly on the ceiling. The upper fluted portion of the ball throws the light downward where it is needed. The lower portion is frosted in order to soften the glare of the filament.

Cellar.—The lights in the cellar are equipped with the flat enameled metal reflectors. These are placed on the ceiling so as to distribute the light well over the walls and floor.

Bedrooms.—For each bedroom a bracket fixture carrying one light in an opal bell reflector is provided. This is placed high enough so that the dresser can be placed directly beneath it, thus furnishing a good light by which to dress. (Fig. 5.) This will also provide a sufficient general illumination if the lamp is inclined about 45° from the horizontal. The lamp should be frosted. An

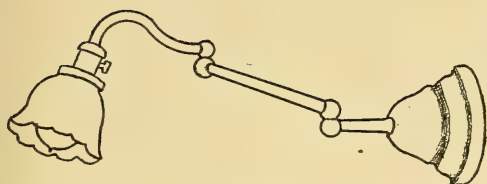


FIG. 3
ADJUSTABLE BRACKET FIXTURE

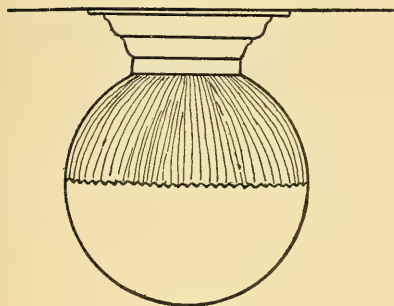


FIG. 4
REFLECTING BALL FOR VERANDA

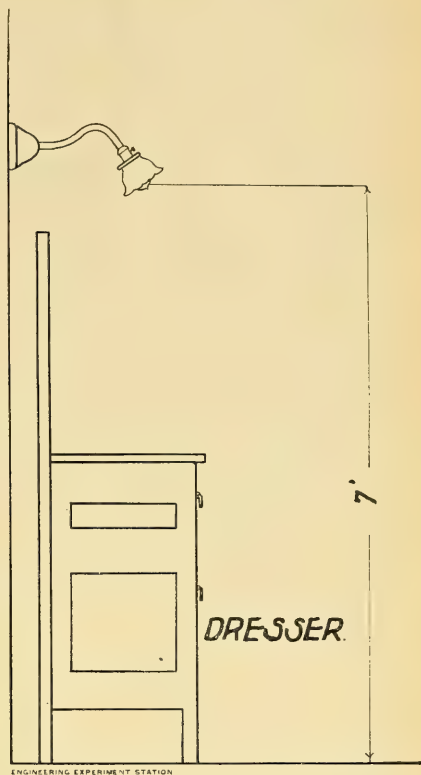


FIG. 5
SKETCH SHOWING GOOD LOCATION OF
BEDROOM FIXTURE

eight candle power carbon lamp is placed in three of the closets. These are simple drop lights suspended about six and one half feet from the floor. No extra length of lamp cord should be provided, otherwise the lamp may be hung upon a hook in contact with some clothing. Then if the lamp is accidentally left lighted a fire is almost sure to occur.

Hall and Bath-room.—Simple, single light pendant fixtures are provided for the second floor hall and the bath room. These are

equipped with opal bell reflectors and are hung about seven and one half feet from the floor. The lamps should be frosted.

DESIGN OF PLANT

Now that we have decided upon the number of lights in each room the next step in the design of our lighting system is to estimate the hours during the day that the lights in each room will be lighted. This will give us an idea of how large our storage battery will have to be to operate the lamps. Of course, the size of the battery will also depend upon how often it is convenient to charge it. Let us assume that we wish our battery to be of sufficient capacity to operate the lights on one charge, the entire day when there is the maximum amount of light used. This will be in the winter when the nights are long and when there is some special occasion that keeps the family up later than usual. We will make out a probable lighting schedule for this day. The schedule is given below. In the column to the right are given the lamp-hours per day. The lamp-hours per day for each room are the number of lights in that room multiplied by the number of hours during the day that they are lighted.

Dining Room: Two lights, on during breakfast and supper,

5:00-6:30 a. m.	}	6 lamp hours
5:30-7:00 p. m.		

Living Room: Three lights, on only after supper,

7:00-10:30 p. m.	10½ lamp hours
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Kitchen: Two lights, on while preparing meals and washing dishes, morning and evening,

5:00-7:30 a. m.	}	10 lamp hours
5:00-7:30 p. m.		

Front Hall: One light

8:00-10:30 p. m.	2½ lamp hours
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Front Porch: One light

7:30-9:00 p. m.	1½ lamp hours
----------------------	---------------

<i>Rear Hall:</i> One light			
5:00-6:00 a. m.....	}	2½ lamp hours	
6:00-7:30 p. m.....			
<i>Bedrooms:</i> Two lights			
5:00-5:30 a. m.....	{	2½ lamp hours	
9:00-9:30 p. m.....			
One light			
10:30-11:00 p. m.....			
		35½ lamp hours	

This gives a total of 35½ lamp hours. Hence, we wish a battery that will operate one lamp approximately 36 hours, with one charge.

Before going further we must become familiar with another unit used in electrical measurements. It is called the ampere and is the unit by which we express current flow. We know that voltage is the electrical pressure or that which tends to make flow an electric current. When we say that a lamp is a 110 volt lamp we mean that it takes 110 volts of electrical pressure to drive sufficient current through the resistance of the lamp filament to heat it hot enough to glow. It does not, let us understand, indicate the amount of electricity that flows. It merely relates to the pressure that causes the current to flow. This current that flows through the filament or any other electrical conductor is measured in amperes. The lamps that we have chosen will allow about one ampere of current to flow through the filament when the pressure of 25 volts is applied, they being 25 volt lamps. We know that to produce continually a pressure of at least 25 volts, 15 storage cells are required. What we want to find out now is how large the cells must be to hold a sufficient charge to let one lamp burn for 36 hours. As we said before, one lamp will permit one ampere of current to flow. Hence the battery will have to be large enough to hold a charge such that one ampere can flow for 36 hours, that is to say, a 36 ampere-hour storage battery, the ampere-hour being the unit of capacity by which manufacturers rate storage batteries. The nearest commercial size of battery to 36 ampere hours is the 40 ampere hour battery. We will, therefore, choose that size of battery.

The "normal rate" of charging a storage battery is the number of amperes of current we must force into the battery to charge it in eight hours from an almost discharged condition. Since

ours is a 40 ampere-hour battery its normal charging rate is 40 divided by 8, or 5 amperes. When there is plenty of time for charging, it is best to charge the battery at the 5 ampere rate. However, if time is lacking it may be charged in a shorter time with a current of 7, 8 or even 9 amperes. This capacity of storage battery will require charging only once per day when there is the heaviest probable load. However, there will not ordinarily be as many lamps burning so many hours of the day, hence the battery will usually not require charging more than once in two days, even in winter. In the summer when the days are long and few lamps are necessary, one charge would be sufficient for a week or more.

Now that the battery is selected, we must decide upon the dynamo with which to charge it. Each cell of battery when fully charged will give a pressure of about 2.6 volts so that the entire 15 cells will give a pressure of 15 times 2.6, or 39 volts. Since in charging a battery the current must flow into the battery in a direction opposite to the flow of current when the battery is discharging, the entire voltage of the battery is opposing the voltage of the dynamo; i. e., the battery is connected to the dynamo so that it tries to drive current through the dynamo, while the dynamo is driving its current at the same time into the same end of the storage battery. If then the dynamo is able to drive any current into the battery when it is almost charged, it must give a higher voltage than the maximum voltage of the battery. Since the maximum voltage of the battery is about 39 volts, then our dynamo should be able to generate about 42 or 43 volts. A 45 volt machine is a regular commercial size, so we will decide upon that voltage machine. The dynamo will have to be able to deliver a current equal to the greatest amount that will probably be used in charging the battery. When charging the battery in a short space of time 8 or 9 amperes may be used, so the current delivered by the dynamo must be at least this amount.

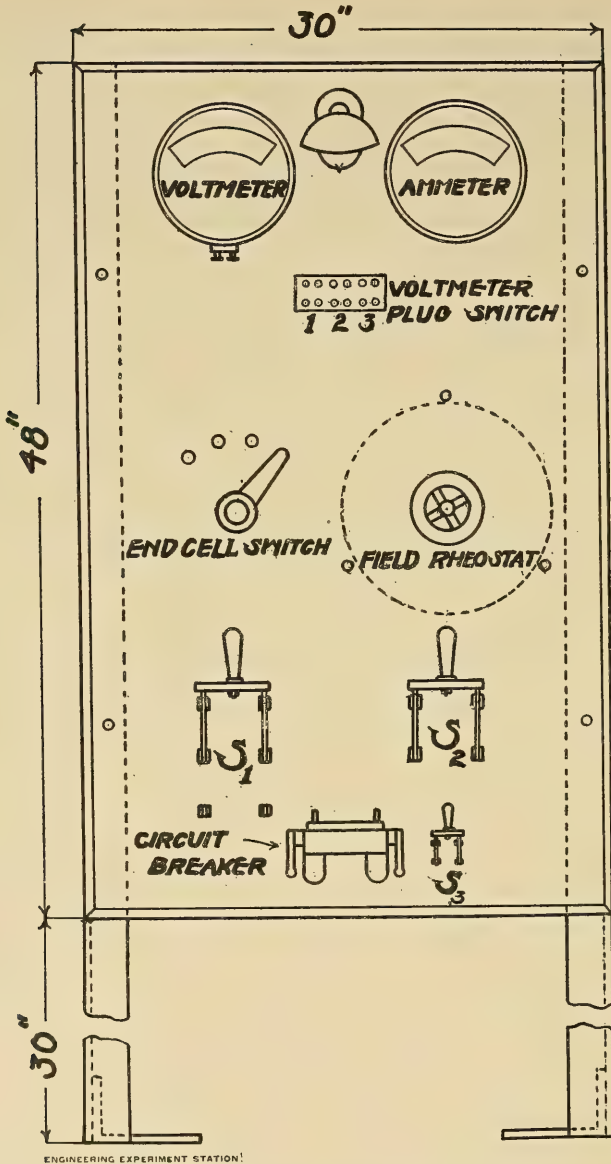
Dynamos are rated by the kilowatts of energy they will produce. As was explained before, a watt of electrical energy is equal to $\frac{1}{746}$ part of a horse-power. A kilowatt is equal to 1000 watts. Hence a kilowatt equals $\frac{1000}{746}$, or about 1.34 horse-power. Now the number of watts of electrical energy produced by a dynamo is equal to the product of the volts of pressure and the amperes of current. The dynamo we need must give at least 9

amperes of current at 45 volts pressure. Hence it must produce 9 times 45, or 405 watts. The nearest commercial size to 405 watts is the $\frac{1}{2}$ kilowatt, or 500 watt size. Hence we shall decide upon a $\frac{1}{2}$ kilowatt machine.

The gasoline engine to drive the dynamo must be able to produce 1.34 times as many horse-power as the dynamo does kilowatts, and besides enough to cover all losses in the dynamo. Since gasoline engines are usually rated rather high, and often on account of some lack of adjustment they do not give their full number of horse-power, it is well to get an engine considerably larger than the size calculated. For the $\frac{1}{2}$ kilowatt dynamo, we should have a 2 horse-power engine.

A switchboard and apparatus with which to control the dynamo and storage battery are next to be selected. An adjustable resistance, called a rheostat, is supplied with the dynamo. This is to enable one to control the voltage of the dynamo so that the battery can be charged either rapidly or slowly. There must be an ammeter to measure the current that is being supplied the battery when charging; a voltmeter to measure the pressure produced by the machine, also that produced by the battery; and the voltage that is supplied the lamps. There should also be a circuit breaker, which is a sort of an automatic switch. Its purpose is as follows. Suppose that the battery is being charged by the dynamo and there is no attendant in the plant. If for any reason the engine stops, the dynamo of course fails to generate any voltage. Since the voltage of the storage battery is now no longer opposed by that of the dynamo, current will flow from the battery and tend to operate the dynamo as an electric motor. To prevent this action the circuit breaker is put in. As soon as for any reason the dynamo stops generating, the circuit breaker automatically opens the circuit and thus prevents current flowing from the battery to the dynamo.

As has been noted before, the storage battery when almost discharged gives only about 1.8 volts per cell, so that to produce the 25 volts necessary to light the lamps to full brilliancy, the entire 15 cells are required. However, when the cells have been fully charged and the charging current is stopped they give about 2.2 volts per cell, so that to light the lamps, all of the cells are not required. Hence a method should be provided to increase gradually the number of cells of battery that are being used, as



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FIG. 6 FRONT VIEW OF SWITCH BOARD

the battery becomes more and more discharged. This will enable us to keep practically constant voltage supplied the lamps so that they will burn at almost their full brilliancy until the battery is discharged. For this purpose we provide an end cell switch.

Since we should be able to obtain a reading of the voltage at three different places and there is only one voltmeter, we must provide a way to switch the voltmeter terminals from one place to another. A plug switch is therefore provided, having three double pairs of holes, or jacks. To the upper pair of each one of these double pairs of jacks are connected the leads coming from the voltmeter. To the lower pair of the first double pair are brought leads from the two terminals of the dynamo; to the second, leads from the terminals of the battery; and to the third, leads from the two sides of the line leading to the lamps. By

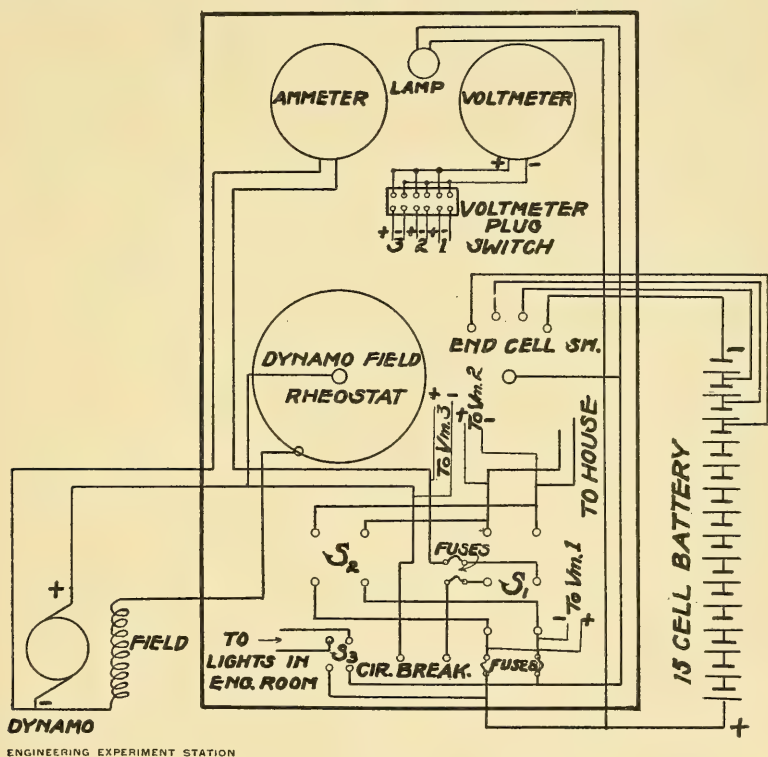
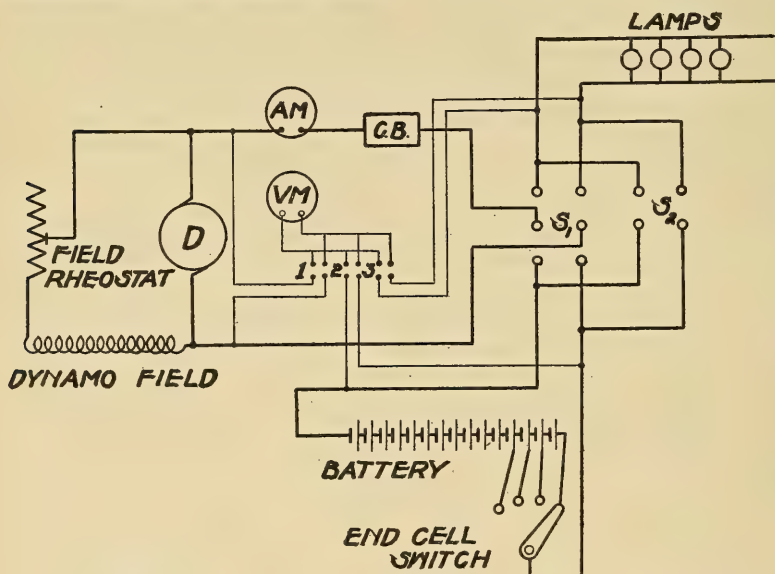


FIG. 7 SWITCH BOARD CONNECTIONS
Shown from Rear

means of a four point plug any one of the three pairs of leads can be switched on the voltmeter.

Two switches are provided,—one double throw switch by which we can throw the dynamo on to the battery for charging, or by throwing it the other way the lights can be operated directly from the dynamo. To operate the lamps directly from the dynamo, the voltage must be reduced to about 26 or 27 volts. Otherwise the lamps will soon be burned out by the excess voltage. By leaving this switch open and closing the other switch, the dynamo circuit is opened and the battery is operating the lights. Of course, by opening the latter switch also the lights are all turned off. Fig. 6 shows a diagram of the switchboard with the apparatus in place. Fig. 7 shows the rear of the board with the connections as they would be made. Fig. 8 shows a schematic diagram of the connections in the power plant. Fig. 9 shows the arrangement of the engine room.



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FIG. 8 WIRING DIAGRAM

AM—Ammeter
 VM—Voltmeter
 CB—Circuit Breaker
 D—Dynamo
 S₁, S₂—Switches

Light Lines are Voltmeter Leads

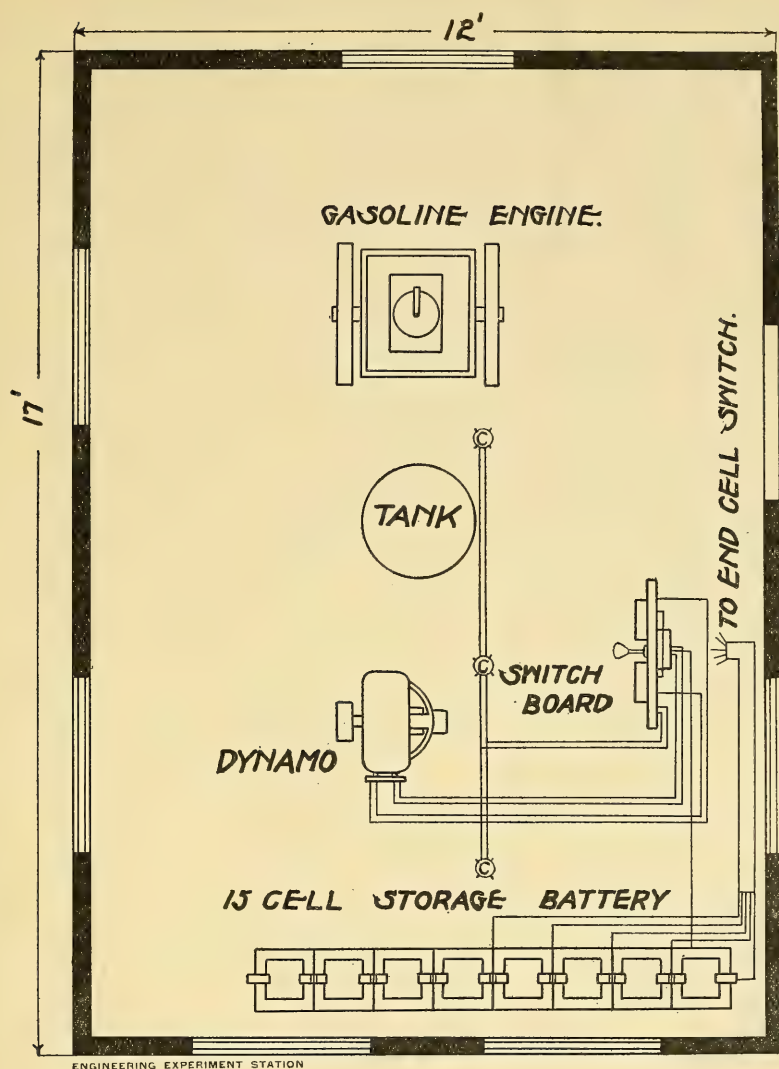


FIG. 9 ENGINE ROOM PLAN

All of the wires leading from the battery to the switchboard and the circuit from the switchboard to the armature terminals of the dynamo, and the one leading from the switchboard to the house should be large enough to carry the maximum current with only a small drop in voltage. For this case, 8 amperes will be

about the maximum current used. Assuming a distance of 200 feet from the switchboard to the cabinet in basement, No. 8 B. and S. gage wire will be large enough to carry the maximum current with only a small loss. Wires of this size are therefore run to the basement and from there directly up to the second floor. From the distribution cabinets on each floor leads are run to each room. These wires need to be no larger than No. 14 since not more than three lights, i. e., three amperes of current are supplied through any circuit.

Ordinarily a plant of this sort would be housed in a building large enough to accommodate all of the machinery of the farm that could be operated by the gasoline engine. Such a building would usually not be fire-proof, so to decrease the fire hazard the gasoline tank should be buried at some distance from the building and the supply pumped to the engine as needed. Where space is limited and where compactness is desired, as would be the case if the plant were used to light a house in town, a fire-proof building would be advisable. Such a building could be built of brick or concrete and should contain a separate compartment for the gasoline tank. The light plant could then be placed near other buildings without danger of fire. If the storage battery were placed in the basement of the house, as may be done, a 10 by 16 ft. building would be of ample size to accommodate the plant.

ESTIMATE OF COST OF PLANT

A good storage battery of the 40 ampere-hour size will cost from \$4.00 to \$5.00 per cell. A quotation of \$4.60 per cell was made by one of the best companies. Since 15 cells are required, the total cost of the battery will be approximately \$70.00.

Gasoline engines can be estimated at about \$60.00 per horsepower, the smallest sizes costing perhaps a little more than this. A two horse-power engine of a good make will cost about \$125.00.

Complete switchboards vary in cost according to the material of the board, i. e., whether of slate or marble and according to the grade of the instruments and switches furnished. A first-class marble switchboard equipped with good instruments will cost approximately \$100.00.

A one-half kilowatt dynamo, shunt wound, 45 volts, of a first-class make can be had for \$65.00.

Seventeen tungsten lamps will be required, each costing \$1.00, making a total of \$17.00.

Fourteen 8 candle power, 25 volt carbon lamps will cost about \$3.00.

1000 feet No. 14 wire	\$12.00
550 feet No. 8 wire	25.00
Cabinet for basement.....	2.85
Cabinet for first floor.....	5.00
Cabinet for second floor	2.85
Porcelain cleats and tubes.....	2.00
18 snap switches	7.20
Labor: 2 men, 8 days, at \$3.00.....	48.00

FIXTURES

<i>Living Room:</i> One 3 light pendant fixture fitted with one Holophane reflector and two stalactites.....	7.00
<i>Dining Room:</i> Two light fixtures with Holophane prismatic bowl reflectors.....	7.00
<i>Front Hall:</i> Single light fixture fitted with Holophane stalactite.....	4.00
<i>Kitchen:</i> Single light fixture fitted with opal bell reflector	2.00
One adjustable bracket fixture	2.00
<i>Veranda:</i> Prismatic reflecting ball in ceiling fixture	2.00
<i>Cellar:</i> Cleat receptacles.....	
Five required.....	1.25
<i>Bedrooms:</i> Single light bracket fixture with opal bell reflector	
Five required.....	6.25
<i>Bath Room:</i> Single light fixture with opal bell reflector	1.50
<i>Hall:</i> Second floor; Single light fixtures with opal bell reflector.....	1.50
<i>Closet and Pantry:</i> Five drop cords and sockets with ceiling rosettes... ..	1.50

This makes a total cost of a little over \$525.00 for the entire outfit and for its installation. Allowing for incidentals and for expenses unprovided for in the estimate the plant will cost not more than \$550.00.

If necessary, the cost of several items of the estimate could be cut down so as to make a cheaper, though a very satisfactory installation. A 2 horse-power gasoline engine can be had on the

market for \$100.00, that will serve our purpose. It will be lighter, will not run quite so steadily and will perhaps not last quite so long, but will do the work in a fairly good manner. The size and consequently the cost of the storage battery can be decreased if we are content to charge oftener. A 30 ampere-hour battery will cost about \$1.00 less per cell than the 40 ampere-hour size. This will decrease the cost of the battery by \$15.00. If one does not care so much for the appearance of the switchboard, a slate panel will do as well as a marble one and would be considerably cheaper. Less expensive instruments can be used so as to cut down the cost of the board to \$80.00 or \$90.00, thus decreasing the cost by about \$15.00. The only method of cutting down the cost of the lighting fixtures that is recommended is to omit the lights in the closets, three of those in the basement and the light from the front porch. This would save about \$5.00 in fixtures, wire, cleats and labor and \$1.50 in lamps. A cabinet for each floor can be made by the wire men cheaper than the price given in the estimate. All three of them should not cost more than \$5.00, decreasing the cost by \$5.50. These changes will decrease the cost \$78.50, making the plant cost a little more than \$470.00.

For anyone who can afford a more expensive plant than the one we have designed there are several changes that can be made. In the apparatus of the engine room, i. e., the engine, battery, dynamo and switchboard there are not many changes that could be made to great advantage. A gasoline engine made especially for driving a dynamo can be bought at an increase of 25 per cent in the price quoted. These are much heavier than the one decided upon and will therefore run much more smoothly and give better satisfaction if lights are ever operated from the dynamo itself. A larger storage battery can be obtained if it is not desirable to charge often. The fixtures used in this design are very simple and inexpensive. If it is desired, much more elaborate and artistic ones can be bought. The dining room fixture may be a fancy art glass dome suspended by a chain. The others may be heavy cast-brass fixtures of any suitable design and costing almost any price that one is willing to pay. Any electric supply house will give quotations upon such fixtures.

SUGGESTIONS ABOUT ORDERING APPARATUS

Before ordering or asking for quotations upon any apparatus, the various steps of the design outlined in this bulletin

should be carried out as carefully as possible and the sizes of the various pieces estimated. Then in asking for quotations, for instance, on the storage battery, it should be stated about what size battery you think will be required. As a check upon the work it would be well to state to the company what size of house the plant is to light, how many lamps in each room and the maximum number of hours each lamp will burn per day. This will give the company information that will enable them to advise you as to the proper size of battery. To the company from whom quotations on the engine are asked it should be stated what size of dynamo it is intended to drive and what machinery, if any, will be operated by the engine at the same time that the dynamo is running and what machinery when the dynamo is not running. The company furnishing the dynamo should know that it is for battery charging, and just how many and what size cells are to be charged from it, and if it is desired to operate the lamps at any time directly from the dynamo. The storage battery companies are almost always in a position to furnish the switchboard and will fit it up according to the design given, if it is sent to them.

OPERATION AND CARE OF APPARATUS

Before attempting to use a plant such as the one just designed, full and complete instructions should be obtained from the various companies for the care and operation of the apparatus supplied by them. This is especially true of the engine and storage battery. All dynamos such as would be used for charging a battery are nearly enough alike so that instructions for the care of one dynamo would apply equally to all. However, gasoline engines and storage batteries differ enough so that explicit instructions should be obtained from the company furnishing them. Some instructions will be given here for the operation of the plant as a whole, for the care of the dynamo and some general directions as to the care and operation of the battery.

It is presumed that a competent person will be obtained to install the apparatus and do the wiring. Such a person can be found in almost any small city, and it will be more satisfactory to employ him than for a novice to attempt to do the work. However, it is seldom that even a man who is perfectly competent to install the machines and do the wiring, knows much about storage batteries. It would be well then to insist, however competent

the man is known to be, that he follow the general directions which will be given regarding the installing and preliminary treatment of the storage batteries.

For the proper operation of the storage battery there will be needed two small pieces of apparatus. One is a hydrometer, which is a small glass instrument similar in appearance to a thermometer. Its purpose is to determine the density of the solution or the electrolyte in which the plates of the battery are immersed. To use one of these hydrometers, all that must be done is to place it in the solution between the plates of the cell, taking care that it floats free of the plates. It will sink so that more or less of its stem is immersed in the liquid. This stem has a graduated scale and the reading taken at the surface of the liquid gives the density, or specific gravity of the electrolyte, that is to say, how many times heavier it is than pure water. The company from whom the battery is bought will furnish one of these hydrometers for about \$1.00 or \$1.25. There will also be needed a small portable voltmeter that will measure up to three volts. This is used for determining the voltage of each individual cell. A suitable instrument of this kind may be had from almost any electrical supply house at prices varying from \$4.00 to \$10.00.

The engine, generator and switchboard should be installed, tested and ready to operate before the storage battery arrives. As soon as the cells arrive they should be unpacked and assembled according to the directions furnished by the company. A strong set of shelves should be built upon which to place the battery, and it should be remembered in building them that nails and bolts are soon corroded and weakened by the acid fumes. These shelves should be arranged so that there is plenty of room around the battery so that the plates can be examined and the voltage and the density of the electrolyte tested. If the cells are placed in two rows, one on top of the other, there should be a space of at least one foot between the top of the lower cell and the bottom of the top shelf. Wooden sand trays and insulators are usually supplied by the company furnishing the battery. Each of these trays is filled with sand and placed upon four of the insulators. The battery jar is then set upon the sand, adjusted so that it is level and has a bearing over the entire bottom. The cells are connected so that the positive plate of one cell is connected to the negative plate of the next through the entire battery. Great

care must be taken to connect the positive plate of the first cell of the battery to the positive terminal of the dynamo. The positive plate of the battery can be distinguished by its dark brown color. The negative has a sort of a yellowish gray color. The positive terminal of the dynamo can be found by tracing back from the switchboard voltmeter. Have the dynamo running and connect the voltmeter to the dynamo terminals so that it reads in the proper direction. Then trace from the voltmeter terminal marked + back to the dynamo. The terminal to which it is traced will be the positive one. Care should be taken that this terminal leads to the end of the battery having the first plate of a brown color.

The acid solution, or electrolyte, should in no case be put into the jars until every thing is ready to begin charging the battery. When all is ready the solution is put in, the engine and dynamo started up, and the rheostat adjusted until the proper charging current is indicated upon the ammeter. As a precautionary measure, it is well before closing the switch to the battery, to adjust the rheostat until the voltmeter reads about 30 volts. Then the switch is closed and the rheostat adjusted until the battery is charging at its 10 hour rate. In the case of the 40 ampere-hour battery, this would be approximately 4 amperes. The electrolyte, which should be obtained ready mixed from the battery company, should show a specific gravity of about 1.17 at the beginning of the charge. The charging should be continued for 10 hours a day for about three days, or until the electrolyte bubbles freely. The specific gravity of the solution and the voltage of the cells will then have reached stationary values of about 1.2 specific gravity and 2.5 volts. During this charge the current should be kept at a constant value of about 4 amperes by the adjustment of the rheostat. The battery may now be used for lighting until discharged down to about 2.00 volts per cell, after which it should be given another thorough charge. This system of moderate discharge and thorough charge should be kept up for two or three cycles of charge and discharge, after which the battery should be in a first-class condition. These are general instructions for almost any type of battery. If explicit instructions for the preliminary treatment of the battery are furnished by the storage battery company, they should be

followed closely instead of the directions given above. The ones given here are for use in case other instructions are lacking.

After the preliminary treatment of the cells, the battery may be charged at any convenient time no matter whether fully discharged or not. If a large engine has been purchased with the idea of operating other machinery at the same time that the dynamo is running, the dynamo may be started and the battery put on charge whenever the other machinery has to be used. Suppose we had decided upon a 3 horse-power engine with the idea in mind of operating a feed grinder at the same time as the dynamo. Then whenever the grinder has to be used it would pay to start the dynamo and charge the battery whether or not it had been discharged down to 1.9 or 2.00 volts per cell.

There are two things that must always be borne in mind in operating the storage battery. The first is that the cells must not under any circumstances be discharged below 1.8 volts per cell and preferably not below 1.9 or 1.95 volts. If discharged to a too low voltage a harmful white sulphate forms upon the lead plates. This sulphate always forms when the plates are being discharged, but if the discharge is not carried too far it is destroyed or reduced when they are charged again. The abnormal sulphate caused by a too low discharge is almost impossible to reduce and it causes the plates to bend or buckle, increases the resistance of the battery and consequently lowers its efficiency and decreases the ampere-hour capacity of the battery. The other precaution is to be sure that the battery gets a thorough charge up to 2.5 or 2.6 volts per cell every week or so. This will reduce the sulphate that has been formed on discharge and will keep the cells in good condition.

The voltage and the density, or specific gravity of the electrolyte of each cell, should be observed at least every two weeks and the battery company notified and asked for advice if either seems to show unusual values. As has been noted before, the voltage per cell should vary from 1.8 volts at the lowest allowable discharge to about 2.6 volts in the fully charged condition dropping to 2.2 or 2.3 volts when the charging current is stopped. The specific gravity should not fall below about 1.18 when discharged nor above 1.24 when fully charged. If the batteries are well cared for the density will usually remain about correct, if they are periodically filled up with pure water to supply the loss

by evaporation. It is best to use nothing but distilled water for the cells, but if no distilled water can be obtained, good, pure filtered rain water may be used.

The battery should be charging at its "normal" rate when filling up the cells with water to make good the losses by evaporation. This causes the water to mix more quickly with the electrolyte and prevents the harmful results caused by having the electrolyte of non-uniform density.

The battery should not be allowed to stand idle after discharge has taken place. Put it on charge as soon as possible after it has reached a voltage of about 1.9 volts per cell.

Sediment should not be permitted to collect in the bottom of the battery jars. If it does collect the cell should be fully charged, removed from the battery, the electrolyte drawn out and the sediment removed.

The color of the plates should be carefully watched, as their color gives a good indication of their condition. When the plates are first set up the negatives are a yellowish gray and the positives dark brown, usually spotted with whitish or reddish gray substances. The spots are sulphate and should disappear when the cell is fully charged. When in good operating condition the positives are dark red, chocolate or plum color, becoming nearly black when charged. If fully discharged, the whitish or reddish gray patches of sulphate appear. The negatives are a sort of a pale slate color that becomes darker as the plates are charged. When the plates are in good condition the surface of the positive plates is soft and the color will rub off on the finger. When in bad condition the surface is usually hard.

Voltmeter reading of the cells should be taken while the battery is discharging in order to get a good idea of the true condition of the cells. Often a cell, when standing idle, will indicate a voltage of perhaps 2.0 volts, but as soon as any current is taken from it, the reading will immediately drop to a much lower value.

The engine room should be heated in the winter to keep the solution from freezing. A low temperature of the electrolyte decreases the capacity of the battery to a marked extent. Hence if the battery is to produce its full rated number of ampere-hours in the winter, the room should be kept at a temperature of not less than 55° or 60° Fahrenheit. In many cases it might be well to have the battery in the cellar of the house. Then the

solution would keep from freezing without the aid of artificial heat. This will, of course, necessitate having the wires to the storage battery much longer than we have estimated. Hence the cost of the wire will be increased somewhat, but the increased cost would probably be justified.

The principal things to watch about the dynamo are the bearings, the commutator and the brushes. Needless to say, the bearings should always be kept supplied with a good quality of oil, and watched to see that the oiling rings rotate freely when the machine is running. The brushes and commutator should be watched carefully. If there is sparking at the brushes they should be examined to see if they fit the commutator perfectly at every point. If not, they should be sandpapered carefully as follows. Take a strip of extra fine sand paper (never emery cloth) a trifle narrower than the commutator is long. Lift up the brush and slip the paper under it with the smooth side next the commutator. Then after letting the brush down and holding the paper snugly against the commutator, carefully draw the paper under the brush in the direction in which the commutator turns when the dynamo is in operation. Lift the brush, remove the paper and repeat the process until the brush fits perfectly. When the commutator is in good condition it will not have a bright metallic copper color, but will have a fine dark luster. If the commutator is bright after operating for a while the brushes probably bear too hard or else there has been sparking at the brushes. Occasionally the commutator should be cleaned by holding a soft rag, slightly oily, upon the commutator while the machine is running. This will lubricate the surface slightly and tend to make the brushes operate smoothly.

COST OF OPERATION

The principal cost of running a plant such as the one designed is the cost of operating a gasoline engine. A two horse-power engine will cost about 5 cents per hour for gasoline running at full load. When the engine is driving the dynamo alone it is giving about .7 horse-power and the cost of gasoline is about 1.8 cents per hour. These figures are assuming gasoline at 18 cents per gallon. The dynamo costs very little to operate; almost the only item is that of oil for the bearings; this is, of course, small. The storage battery requires no supplies except that occasionally

some sulphuric acid will have to be added to the electrolyte of those cells whose specific gravity has fallen low. The acid costs only about 5 or 6 cents a pound and only a small quantity is needed so this item is almost negligible. Depreciation is the most important item in storage battery operation and this depends altogether upon the treatment of the battery. It matters not how good or how poor a cell may be, careless treatment will reduce its life of useful service to a few months. The cost of maintenance or making good this depreciation is practically that of renewing the plates. With careful use the positive plates of a battery such as has been selected for this design will probably need renewal in 4 or 5 years and the negatives in 8 or 10 years. This will make the average annual cost of maintaining the battery \$8.00 or \$10.00 per year.

GLOSSARY OF TECHNICAL TERMS

Acid Solution. The mixture of sulphuric acid and distilled water in which the plates of the storage battery are immersed.

Ammeter. An instrument for measuring the amount in amperes of electric current flowing in the circuit.

Ampere. The unit of current flow in electricity. It is the current that will flow when a pressure of 1 volt is applied to a circuit having a resistance of 1 ohm.

Ampere-Hour. The number of amperes flowing in a circuit multiplied by the number of hours that it flows.

Armature. That part of the dynamo in which the voltage is generated. Usually the rotating part of the dynamo.

Bracket Fixture. A lighting fixture which is fastened to the wall in the manner of a bracket. See Fig. 3.

Brilliancy of Illumination. The brightness or intensity of illumination. It depends upon the candle power and the distance of the surface illuminated from the light giving source.

Brushes. The small carbon blocks on a dynamo which collect the current from the rotating commutator.

Cabinet. A box or cabinet from which is made the distribution of wires to the different rooms on each floor.

Capacity. (Of battery). The number of ampere-hours of charge the storage battery is designed to receive. (Of dynamo). The number of kilowatts the dynamo is designed to

generate. (Of engine). The number of horse-power the engine is designed to produce.

Ceiling Rosette. A device made of porcelain by means of which connections are made from a drop cord through the ceiling to the wires above.

Cell. One complete unit of a storage battery consisting of jar, plates, electrolyte, sand tray and insulators.

Circuit Breaker. An automatic switch for opening battery charging circuit when for any reason, the dynamo fails to deliver current.

Cleat. A small porcelain device for fastening and insulating current carrying wires.

Cleat Receptacle. A combination of a cleat and receptacle for holding an incandescent lamp.

Commutator. The rotating segmented copper portion of the dynamo from which the current is collected by means of the brushes.

Conductor. Any substance which allows electric current to flow through it. The insulated wire which is used to carry the current.

Current. The amount of electricity that flows. It is measured in amperes.

Density of Electrolyte. The specific gravity of the electrolyte. The number of times a given quantity of electrolyte is heavier than the same quantity of pure water.

Diffusing Globe. A globe that breaks the light up into fine rays, thus softening it to the eyes and removing the dazzling and glaring effect of the light.

Diffusion. The process of breaking the light up into fine rays, as by means of a diffusing globe.

Distilled Water. Water that has been condensed from steam.

Double Throw Switch. A switch arranged so as to enable one to throw either of two pairs of wires into circuit. See "S" in Fig. 6.

Drop Cord. A pair of flexible rubber insulated wires fitted with a lamp socket. They are used where a very cheap sort of a lighting fixture is desired.

Dynamo. A machine for generating electrical power.

Electrolyte. The acid solution. A mixture of sulphuric acid and water in which the plates of the storage battery are immersed.

End Cell. A cell of storage battery which is arranged so that it can be cut into or out of the circuit, so as to regulate the voltage of the battery.

End Cell Switch. The switch by means of which the end cells are cut into or out of the circuit.

Fahrenheit. The scale into which the thermometers ordinarily in use are divided.

Filament. The light-giving part of an incandescent electric lamp.

Generator. A machine for generating electrical power. A dynamo.

Holophane Globe or Shade. A globe or shade made by the Holophane Glass Co. They are made of clear glass, but have flutings or prisms that deflect and diffuse the light.

Horse-Power. The unit of mechanical power. It requires one horse-power to raise 33,000 pounds one foot high in one minute or the equivalent. A horse-power equals 746 watts of electrical power.

Hydrometer. An instrument for measuring the density of specific gravity of a liquid.

Incandescent Electric Lamp. An electric lamp in which a conducting filament is heated by an electric current to a temperature sufficiently high to give off light.

Insulator. A substance that will not permit the passage of an electric current. A sort of glass knob used to insulate the storage battery from the ground.

Intensity of Light. The number of candle power of light produced by a lamp divided by the area of the light-giving source, i. e., in an electric lamp, the area of the filament.

Kilowatt. Equal to 1000 watts or 1.34 horse-power.

Lamp Cord. A flexible, twisted, two conductor wire used in making drop and extension cords.

Lamp-Hours. The number of lamps multiplied by the number of hours during which they burn.

Negative Plate. The grayish colored plate of the storage battery.

Negative Terminal. The terminal of the dynamo or battery to which the current returns after passing through the circuit.

Normal Rate of Charge. The number of amperes which must be forced into a battery to charge it in 8 hours from a nearly discharged condition.

Pendant Fixture. A lighting fixture which is hung from the ceiling. See Fig. 2.

- Plug Switch.* A switch in which electrical connection is made by placing a metallic plug into a metallic jack or hole.
- Positive Plate.* The brown or chocolate-colored plate of a storage battery.
- Positive Terminal.* The terminal of a dynamo or battery from which the current flows to the circuit.
- Pressure. Voltage.* That which tends to cause an electric current to flow.
- Prismatic Reflecting Globes.* Globes made of clear glass which diffuse and reflect the light by means of prisms and flutings on the surface.
- Resistance.* The opposition any conductor presents to the flow of an electric current. It is measured in ohms. 1000 feet of copper wire .1 of an inch in diameter has a resistance of 1 ohm.
- Rheostat.* An adjustable resistance used for varying the current in a circuit.
- Snap Switch.* A small spring switch operated by turning a small button or key.
- Socket.* A device for holding and making electrical connection to the filament of an incandescent lamp.
- Specific Gravity. Density.* The number of times any substance is heavier than pure water.
- Storage Battery.* An apparatus consisting of prepared lead plates immersed in dilute sulphuric acid by means of which electric power is stored.
- Sulphate.* A white substance that sometimes forms on the plates of a storage battery.
- Sulphuric Acid.* The acid whose chemical formula is H_2SO_4 . It is used diluted as the electrolyte for a storage battery.
- Switchboard.* The slate or marble panel upon which measuring instruments, controlling switches, rheostat, etc., necessary for the operation of the plant, are mounted.
- Tubes.* Cylindrical tubes made of porcelain; used for insulating a current-carrying wire where it passes through a hole in the wall, partition, etc.
- Tungsten Filament.* A filament for an incandescent lamp made from the rare metal, tungsten.

Volt. The unit of electrical pressure or voltage. It is equal to about half the pressure produced by an ordinary dry battery.

Voltage. Difference of electrical pressure or that which tends to make current flow.

Voltmeter. An instrument for measuring voltage.

Watt. The unit of electrical power. Equal to $\frac{1}{746}$ part of a horsepower. Watts equal volts times amperes.

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BULLETIN NO. 26

HIGH STEAM-PRESSURES IN LOCOMOTIVE SERVICE

(A REVIEW OF A REPORT TO THE CARNEGIE INSTITUTION OF
WASHINGTON)

BY

W. F. M. GOSS



UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

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THE Engineering Experiment Station was established by action of the Board of Trustees December 8, 1903. It is the purpose of the Station to carry on investigations along various lines of engineering and to study problems of importance to professional engineers and to the manufacturing, railway, mining, constructional, and industrial interests of the State.

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PREFACE

The *Report on High Steam-Pressures in Locomotive Service*, issued by the Carnegie Institution of Washington as Serial No. 66, is a publication of 144 pages dealing with a research which was carried on in the laboratory of Purdue University during the writer's connection with that University. It illustrates and describes the locomotive and other apparatus employed, and presents in tabulated and graphical form the full record of observed and derived results. In this Review, the text of the Report has been freely quoted, and the conclusions and arguments by which they are sustained appear as given in the original publication. The Review, therefore, takes the form of a résumé of the research and its results, the complete record of which is available elsewhere.

In the editorial work incident to the preparation of this Review, Mr. Paul Diserens has had an important share.

W. F. M. G.

December, 1908.

UNIVERSITY OF ILLINOIS

ENGINEERING EXPERIMENT STATION

BULLETIN No. 26

SEPTEMBER 1908

HIGH STEAM-PRESSURES IN LOCOMOTIVE SERVICE

(A Review of a Report to the Carnegie Institution of Washington)

By W. F. M. GOSS, DEAN OF THE COLLEGE OF ENGINEERING AND
DIRECTOR OF THE SCHOOL OF RAILWAY ENGINEERING AND
ADMINISTRATION

INTRODUCTION

A SUMMARY OF CONCLUSIONS

The results of the study concerning the value of high steam-pressures in locomotive service, the details of which are presented in succeeding pages, may be summarized as follows:

1. The results apply only to practice involving single-expansion locomotives using saturated steam. Pressures specified are to be accepted as running pressures. They are not necessarily those at which safety valves open.

2. Tests have been made to determine the performance of a typical locomotive when operating under a variety of conditions with reference to speed, power, and steam-pressure. The results of one hundred such tests have been recorded.

3. The steam consumption under normal conditions of running has been established as follows:

Boiler pressure 120 lb., steam per indicated horse-power hour 29.1 lb.

Boiler pressure 140 lb., steam per indicated horse-power hour 27.7 lb.

Boiler pressure 160 lb., steam per indicated horse-power hour 26.6 lb.

Boiler pressure 180 lb., steam per indicated horse-power hour 26.0 lb.

Boiler pressure 200 lb., steam per indicated horse-power hour 25.5 lb.

Boiler pressure 220 lb., steam per indicated horse-power hour 25.1 lb.

Boiler pressure 240 lb., steam per indicated horse-power hour 24.7 lb.

4. The results show that the higher the pressure, the smaller the possible gain resulting from a given increment of pressure. An increase of pressure from 160 to 200 lb. results in a saving of 1.1 lb. of steam per horse-power hour, while a similar change from 200 lb. to 240 lb. improves the performance only to the extent of 0.8 lb. per horse-power hour.

5. The coal consumption under normal conditions of running has been established as follows:

Boiler pressure 120 lb., coal per indicated horse-power hour	4.00 lb.
Boiler pressure 140 lb., coal per indicated horse-power hour	3.77 lb.
Boiler pressure 160 lb., coal per indicated horse-power hour	3.59 lb.
Boiler pressure 180 lb., coal per indicated horse-power hour	3.50 lb.
Boiler pressure 200 lb., coal per indicated horse-power hour	3.43 lb.
Boiler pressure 220 lb., coal per indicated horse-power hour	3.37 lb.
Boiler pressure 240 lb., coal per indicated horse-power hour	3.31 lb.

6. An increase of pressure from 160 to 200 lb. results in a saving of 0.16 lb. of coal per horse-power hour, while a similar change from 200 to 240 lb. results in a saving of but 0.12 lb.

7. Under service conditions, the improvement in performance with increase of pressure will depend upon the degree of perfection attending the maintenance of the locomotive. The values quoted in the preceding paragraphs assume a high order of maintenance. If this is lacking, it may easily happen that the saving which is anticipated through the adoption of higher pressures will entirely disappear.

8. The difficulties to be met in the maintenance both of boiler and cylinders increase with increase of pressure.

9. The results supply an accurate measure by which to determine the advantage of increasing the capacity of a boiler. For the development of a given power, any increase in boiler capacity brings its return in improved performance without adding to the cost of maintenance or opening any new avenues for incidental losses. As a means to improvement, it is more certain than that which is offered by increase of pressure.

10. As the scale of pressure is ascended, an opportunity to further increase the weight of a locomotive should in many cases find expression in the design of a boiler of increased capacity rather than in one for higher pressures.

11. Assuming 180 lb. pressure to have been accepted as standard, and assuming the maintenance to be of the highest order, it

will be found good practice to utilize any allowable increase in weight by providing a larger boiler rather than by providing a stronger boiler to permit higher pressures.

12. Wherever the maintenance is not of the highest order, the standard running pressure should be below 180 lb.

13. Wherever the water which must be used in boilers contains foaming or scale-making admixtures, best results are likely to be secured by fixing the running pressure below the limit of 180 lb.

14. A simple locomotive using saturated steam will render good and efficient service when the running pressure is as low as 160 lb.; under most favorable conditions, no argument is to be found in the economic performance of the engine which can justify the use of pressures greater than 200 lb.

HIGH STEAM-PRESSURES IN LOCOMOTIVE SERVICE

I. THE RESEARCH AND THE MEANS EMPLOYED IN ITS ADVANCEMENT

1. *Steam-Pressures in Locomotive Service.*—For many years past there has been a gradual but nevertheless a steady increase in the pressure of steam employed in American locomotive service. Between 1860 and 1870 a pressure of 100 lb. per sq. in. was common. Before 1890 practice had carried the limit beyond 150 lb. At the present time 200 lb. is most common, but an occasional resort to pressures above this limit suggests a disposition to exceed it.

High steam-pressure does not necessarily imply high power. It is but one of the factors upon which power depends. The forces which are set up by the action of the engine are as much dependent upon cylinder volume as upon boiler-pressure, and when the pressure is once determined the cylinders may be designed for any power. The limit in any case is to be found when the boiler can no longer generate sufficient steam to supply them. The relation between pressure and power is therefore only an indirect one. But anything which makes the boiler of a locomotive more efficient in the generation of steam, or the engines more economical in their use of steam, will permit an extension in the limit of power. If, for example, it can be shown that higher steam-pressure promotes economy in the use of steam, higher steam-pressure at once becomes an indirect means for increasing power. The fact to be emphasized is that an argument in favor of higher steam-pressures must concern itself with the effects produced upon the economic performance of the boiler or engine.

2. *Preparations for an Experimental Study.*—In view of the facts stated, and with the hope of ascertaining a logical basis from which to determine what the pressure should be for a sim-

ple locomotive, using saturated steam, it was long ago determined to undertake an experimental study of the problem upon the testing plant of Purdue University. A few experiments involving the use of different steam-pressures in locomotive service were made at Purdue as early as 1895, but as the boiler of the locomotive then upon the testing-plant was not capable of withstanding pressures greater than 150 lb., these early tests were limited in their scope.¹ The matter was, however, regarded as of such importance that in designing a new locomotive for use upon the plant, a pressure of 250 lb. was specified—a limit which then was and still is considerably in advance of practice. Thus equipped, an elaborate investigation was outlined, involving a series of tests under six different pressures, representing a sufficient number of different speeds and cut-offs to define the performance of the locomotive under a great range of conditions. But the expense of operating the locomotive under very high steam-pressures proved to be so great that the limited funds which could be devoted to the operations of the laboratory, in combination with the demands of students, which could be most easily satisfied by work under lower pressures, made it impracticable for a time to proceed with the work. A grant from the Carnegie Institution of Washington was announced late in the fall of 1903. The first test in the Carnegie series was run February 15, 1904, and the last August 7, 1905. A registering counter attached to the locomotive shows that between these dates the locomotive drivers made 3,113,333 revolutions, which is equivalent to 14,072 miles.

3. *The Tests.*—The tests outlined included a series of runs for which the average pressure was to be, respectively, 240, 220, 200, 180, 160, and 120 lb., a range which extends far below and well above pressures which are common in present practice. It was planned to have the tests of each series sufficiently numerous to define completely the performance of the engine when operated under a number of different speeds and when using steam in the cylinders under several degrees of expansion. As far as practicable, each test was to be of sufficient duration to permit the efficiency of the engine and boiler to be accurately determined, but where this could not be done cards were to be taken. A precise statement of the conditions under which, in the development of

¹ Results of these tests will be found published in *Locomotive Performance*, John Wiley & Sons.

this plan, the tests were actually run, is set forth diagrammatically in Fig. 1 to 6 accompanying, in which vertical distances represent speed, and horizontal distances the point of cut-off as determined by the notch occupied by the latch of the reverse lever, counting from the center forward. Each complete circle in these diagrams represents an efficiency test, and each dotted circle, a shorter test under conditions involving the development of power in excess of that which could be constantly sustained. The numerals within the circles refer to the laboratory numbers by which the several tests are identified.

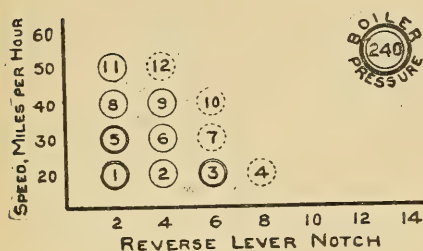


FIG. 1

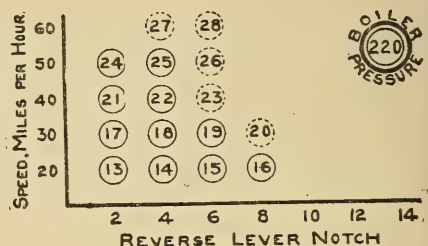


FIG. 2

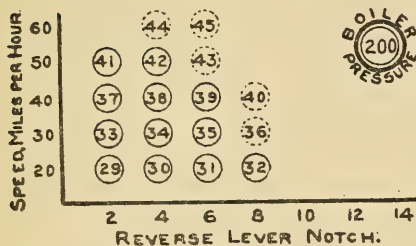


FIG. 3

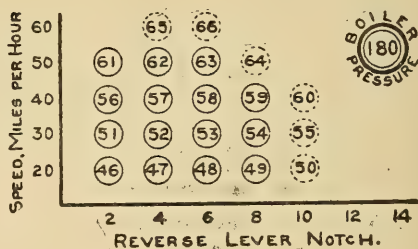


FIG. 4

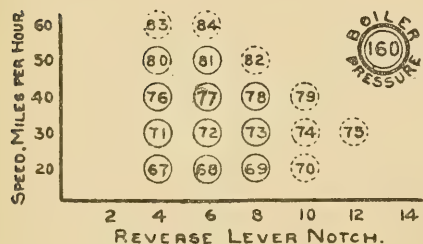


FIG. 5

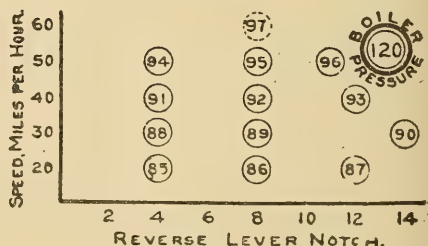


FIG. 6

4. *The locomotive upon which the tests were made is that regularly employed in the laboratory of Purdue University, where it is known as Schenectady No. 2. It was ordered of the Schenectady Locomotive Works in 1897. In selecting a second locomotive which should serve the purposes of the Purdue testing-plant, it was decided to have the boiler of substantially the same capacity as that of the locomotive previously employed in the laboratory and which in later years has been known as Schenectady No. 1. In some other respects the new locomotive differed from its predecessor. Its boiler was designed to operate under pressures as high as 250 lb., a limit which was then 25 per cent higher than the maximum employed in practice. Horizontal seams are butt-jointed with welt strips inside and out, and are sextuple-riveted. The design of its cylinders and saddle is such as readily to permit the conversion of the simple engine into a two-cylinder compound. The driving-wheels of the new locomotive are of larger diameter than those of Schenectady No. 1.*

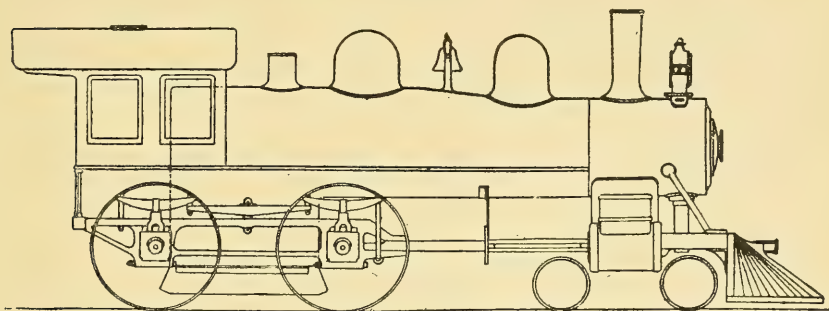


FIG. 7 OUTLINE ELEVATION OF LOCOMOTIVE

The principal characteristics of the locomotive are as follows:

Type.....		4-4-0
Total weight.....	pounds	109 000
Weight on four drivers.....	pounds	61 000
Valves: type, Richardson balanced		
Maximum travel.....	inches	6
Outside lap.....	inches	1 $\frac{1}{8}$
Inside lap.....	inches	0
Ports:		
Length.....	inches	12.0
Width of steam port.....	inches	1.5
Width of exhaust port.....	inches	3.0
Total wheel base.....	feet	23

Rigid wheel base.....	feet	8.5
Cylinders:		
Diameter.....	inches	16
Stroke.....	inches	24
Drivers, diameter front tire.....	inches	69.25
Boilers, (style, extended wagon-top:)		
Diameter of front end.....	inches	52
Number of tubes.....		200
Gage of tube.....		12
Diameter of tube.....	inches	2
Length of tube.....	feet	11.5
Length of fire-box.....	inches	72.06
Width of fire-box.....	inches	34.25
Depth of fire-box.....	inches	79.00
Heating-surface in fire-box.....	square feet	126.0
Heating-surface in tubes, water side	square feet	1196.00
Heating-surface in tubes, fire side.....	square feet	1086.00
Total heating-surface including water side of tubes.....	square feet	1322.00
Total heating-surface including fire side of tubes.....	square feet	1212.00
Total heating-surface, value accepted for use in all calculations.....	square feet	1322.00
Ratio of total heating-surface based on water side of tubes to that based on fire side of tubes.....		1.091
Grate area.....	square feet	17.00
Thickness of crown-sheet.....	inches	$\frac{7}{16}$
Thickness of tube sheet.....	inches	$\frac{9}{16}$
Thickness of side and back sheets.....	inches	$\frac{8}{8}$
Diameter of stay-bolts.....	inches	1
Diameter of radial stays.....	inches	1 $\frac{1}{8}$
Driving-axle journals:		
Diameter.....	inches	7 $\frac{1}{2}$
Length.....	inches	8 $\frac{1}{2}$

5. *An Alternative for Higher Steam-pressures.*—Previous publications from the Purdue laboratory have shown the possibility under certain conditions of finding a substitute for very high boiler-pressures in the adoption of a boiler of larger capacity, the pressure remaining unchanged. If, for example, in designing a new locomotive, it is found possible to allow an increase of weight in the boiler, as compared with that of some older type of machine, it becomes a question as to whether this possible increase in weight should be utilized by providing for a high-pressure or for an increase in the extent of heating-surface. The results of tests, supplemented by facts concerning the weight of boilers designed for different pressures and for different capacities, supply the data necessary for an analysis of this question. Such an analysis is presented elsewhere. (See Chapters VI and VII.)

II. DIFFICULTIES IN OPERATING UNDER HIGH-PRESSURES

6. *The Work with the Experimental Locomotive* has shown that those difficulties which in locomotive operation are usually ascribed to bad water increase rapidly as the pressure is increased. The water-supply of the Purdue laboratory contains a considerable amount of magnesia and carbonate of lime. When used in boilers carrying low pressure there is no great difficulty in washing out practically all sediment. The boiler of the first experimental locomotive, *Schenectady No. 1*, which carried but 140 lb. and was run at a pressure of 130 lb., after serving in the work of the laboratory for a period of six years, left the testing-plant with a boiler which was practically clean. Throughout its period of service this boiler rarely required the attention of a boiler-maker to keep it tight. Water from the same source was ordinarily used in the boiler of *Schenectady No. 2*, which carried a pressure of 200 lb. or more. It was early found that this boiler operating under the higher pressure frequently required the attention of a boiler-maker. After having been operated for no more than 30,000 miles, cracks developed in the side-sheets, making it impossible to keep the boiler tight, and new side-sheets were applied. In operating under pressures as high as 240 lb., the temperature of the water delivered by the injector was so high that scale was deposited in the check-valve, in the delivery-pipe, and in the delivery-tube of the injector. Under this pressure, with the water normal to the laboratory, the injectors often failed after they had been in action for a period of two hours. The interruptions of tests through failure of the injector, and through the starting of leaks at stay-bolts, as the tests proceeded, became so annoying that, as a last resort, a new source of water supply was found in the return tank of the University heating-plant. This gave practically distilled water, and its use greatly assisted in running the tests at 240 lb. pressure.

Probably some of the difficulties experienced in operating under very high steam-pressures were due to the experimental character of the plant, and would not appear after practice had become committed to the use of such pressures by a gradual process of approach, but the results are clear in their indication that the problem of boiler maintenance, especially in bad-water districts, will become more complicated as pressures are further in

creased. Since, taking the country over, there are few localities where locomotives can be furnished with pure water, the conclusion stated should be accepted as rather far-reaching in its effect.

The tests developed no serious difficulties in the lubrication of valves and pistons under pressures as high as 240 lb., though this could not be done with the grade of oil previously employed.

With increase of pressure any incidental leakage, either of the boiler or from cylinders, becomes more serious in its effect upon performance. In advancing the work of the laboratory, every effort was made to prevent loss from such causes, and tests were frequently thrown out and repeated because of the development of leaks of steam around piston and valve rods, or of water from the boiler. Notwithstanding the care taken, it was impossible under the higher pressures to prevent all leakage, and the best that can be said for the data under these conditions is that they represent results which are as free as practicable from irregularities arising from the causes referred to; that is, as far as leakage may affect performance, the results of the laboratory tests may safely be accepted as a record of maximum performance.

In concluding this brief review of the difficulties encountered in the operation of locomotives under very high steam-pressures, the reader is reminded that an increase of pressure is an embellishment to which each detail in the design of the whole machine must give a proper response. A locomotive which is to operate under such pressure will need to be more carefully designed and more perfectly maintained than a similar locomotive designed for lower pressure; and much of that which is crude and imperfect, but nevertheless serviceable in the operation of locomotives using a lower pressure, must give way to a more perfect practice in the presence of the higher pressure.

III. BOILER PERFORMANCE

7. *The Performance of the Boiler.*—The pounds of water evaporated per pound of coal plotted in terms of rate of evaporation is shown for each of the several pressures in Fig. 8. The equations representing the performance of the boiler and furnace as established by these lines are:

$$E = 11.040 - .221 H, \text{ when pressure is 240}$$

$$E = 11.310 - .221 H, \text{ when pressure is 220}$$

$$E = 11.373 - .221 H, \text{ when pressure is 200}$$

$$E = 11.469 - .221 H, \text{ when pressure is 160}$$

$$E = 11.357 - .221 H, \text{ when pressure is 120}$$

where E is the number of pounds of water evaporated from and at 212° per pound of coal, and H is the number of pounds of water evaporated from and at 212° per sq. ft. of heating-surface per hour. The area of heating-surface employed is based upon the interior surface of the fire-box and the exterior surface of the tubes. In determining the position of the lines represented by these equations certain conventions were adopted. These, and the reasons underlying them, may be described as follows:

The only difference in the running conditions applying to the tests of each series is that of pressure, and as the terms employed in plotting the several diagrams are the same, it is evident that the differences in performance are only such as may result from the difference in pressure. Since the quantities are in terms of equivalent evaporation, the differences can not be great. Accepting this view, it was first sought to determine the slope of the lines for the several groups. This was done by plotting upon a single sheet all the points, eight in number, available for the series at 240 lb. together with eight points selected as fairly representative from each of the other series, making forty points in all. The result is shown in Fig. 9. Points thus plotted were divided into two groups, one representing the lower rates of combustion, and the other representing the higher rates, the points being so chosen that each group contained four points from each of the several series. The ordinates and abscissæ for points of each group were then determined, and the several values thus obtained averaged. The final results were then plotted, giving the points shown by the circles inclosing a cross (Fig. 9).

The equation from the line drawn through these points is

$$E = 11.305 - 0.221 H$$

The line thus found (Fig. 9) may fairly be assumed to represent the slope of the mean line of any number of points which for purposes of comparison may be selected from the larger group.

In determining, therefore, the location of the mean lines (Fig. 8), the abscissæ and ordinates of all points were averaged and

the results plotted. Through the derived point a line is drawn having the slope already found; that is, the mean line of Fig. 9.

8. *Effect of Changes in Steam-pressure upon the Evaporative Efficiency of the Boiler.*—The generation of steam at a pressure of 120 lb. involves a temperature of the water which is 50° less than that which must be dealt with in generating steam at a pressure of 240 lb., and in general it has been assumed that any increase in boiler-pressure necessarily results in some loss of evaporative efficiency. It has been known that for the small ranges of pressure common in stationary practice this difference is not great, but the facts have not been established with reference to locomotive performance or for ranges as great as those covered by the experiments under consideration in any service.

The performance of the boiler experimented upon under a range of pressure varying from 240 to 120 lb. may be seen by comparing the mean curves already developed (Fig. 8). This diagram shows that the lowest efficiency is obtained with the highest pressure and that with one exception the lines representing performance under different pressures fall in order, inversely with the pressure. The exception is to be found in the line representing performance at 120 lb. pressure. This line falls low, a condition which may be explained by the fact that the spark and cinder losses for these tests are known to have been excessive. The mean line located from 40 points, representing all pressures (Fig. 9), will represent any of the lines of Fig. 8 with an error not greater than 0.2 lb.

The results clearly define four general facts, which may be stated as follows:

(a). The evaporative efficiency of a locomotive boiler is but slightly affected by changes in pressure.

(b). Changes in steam-pressure between the limits of 120 lb. and 240 lb. will produce an effect upon the efficiency of the boiler which will be less than 0.5 lb. of water per pound of coal.

(c). The equation $E = 11.305 - 0.221 H$ represents the evaporative efficiency of the boiler of locomotive *Schenectady No. 2* when fired with Youghiogheny coal for all pressures between the limits of 120 lb. and 240 lb. with an average error for any pressure which does not exceed 2.1 per cent.

9. *Smoke-box Temperatures.*—The results of the tests show that in all cases the temperature of the smoke-box gases increases as the rate of evaporation increases. Plotted diagrams showing

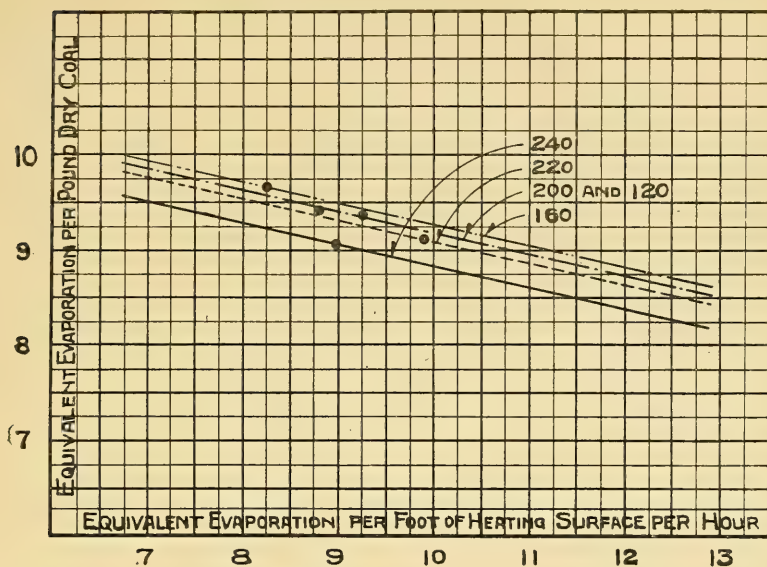


FIG. 8 EVAPORATION PER POUND OF COAL UNDER DIFFERENT CONDITIONS OF PRESSURE

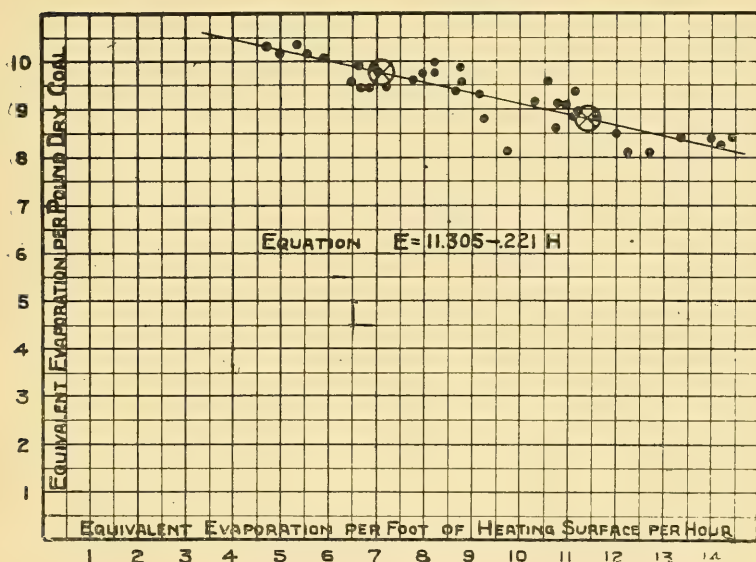


FIG. 9 EVAPORATION PER POUND OF COAL UNDER ALL CONDITIONS OF PRESSURE

the exact relationship indicate a marked similarity for all pressures; all have the same slope and if superimposed they would fall very closely together.

Thus, they show that when the rate of evaporation is 9 lb. per ft. of heating-surface per hour, the smoke-box temperature for all pressures is between the limits of 700° and 730° F. There are but four results for a pressure of 240 lb., in comparison with eight or more for other pressures. If the results from the tests at 240 lb. pressure be omitted it will be found that those remaining, which represent a range of pressure from 220 lb. to 120 lb., are nearly identical. This is best shown by the equations of the curves in question, which are given in Table 1.

TABLE 1
SMOKE-BOX TEMPERATURES UNDER DIFFERENT PRESSURES

Boiler-pressure pounds	Equations
220	$T = 496.3 + 25.66 H$
200	$T = 491.0 + 25.66 H$
160	$T = 487.7 + 25.66 H$
120	$T = 478.9 + 25.66 H$
Average	$T = 488.5 + 25.66 H$

The average of the several equations represents the average of any of the several groups of results obtained under different pressures, with an error which in no case exceeds 10° F., or 2 per cent.

Again, the equations show that the effect of increasing the pressure from 120 lb. to 220 lb. is to increase the smoke-box temperature 17°; that is, an increase of pressure of nearly 100 per cent results in an increase of smoke-box temperature of approximately 3.5 per cent.

In the preceding statements is to be found an explanation of the constancy in the evaporative efficiency of the boiler under different steam-pressures. The fact seems to be that the water in the boiler is about as effective in absorbing the heat of the gases

when its temperature is 400° (240 lb. pressure) as when its temperature is but 350° (120 lb. pressure).

The data sustain the following conclusions:

(a). The smoke-box temperature falls between the limits of 590° F. and 850° F., the lower limit agreeing with a rate of evaporation of 4 lb. per ft. of heating-surface per hour and the latter with a rate of evaporation of 14 lb. per ft. of heating-surface per hour.

(b). The smoke-box temperature is so slightly affected by changes in steam-pressure as to make negligible the influence of such changes in pressure for all ordinary ranges.

(c). The equation $T = 488.5 + 25.66 H$, where T is the temperature of the smoke-box expressed in degrees F., and H is pounds of water evaporated from and at 212° per. ft. of heating-surface per hour, possesses a high degree of accuracy.

10. *Draft*.—The term “draft,” as herein employed, represents a reduction of pressure as compared with that of the atmosphere expressed in inches of water. The draft was observed at three different points between the ash-pan and the stack. These were the smoke-box in front of the diaphragm, the smoke-box back of the diaphragm, and the fire-box. At each of these points connection was made with a U-tube containing water. The results for each different steam-pressure vary but little so that those representing the draft as affected by rate of evaporation for any one pressure, for example, 160 (Fig. 10), are fairly representative of the entire exhibit. Referring to Fig. 10, the solid points represent the draft in the smoke-box in front of the diaphragm; the crosses, the draft behind the diaphragm; and the circles, the draft in the fire-box. Expressing the results in other terms, it appears that vertical distances between the highest curve and the intermediate represent the resistance of the diaphragm; vertical distances between the intermediate and the lowest curve, the resistance of the tubes; and vertical distances between the lowest curve and the axis, the resistance of the ash pan, the grate, and the fire upon it. Values under this curve are a close approach to the effective draft. In general, draft values vary greatly with the conditions at the grate. A thin, clean fire results in comparatively low draft values throughout the system, while a thick fire, or one which is choked by clinkers, leads to the reverse re-

sults. It is for this reason that individual points representing draft sometimes vary widely from the mean of all results.

When the rate of evaporation is 10 lb. per ft. of heating-surface per hour, the draft in front of the diaphragm is approximately 4 inches for all pressures.

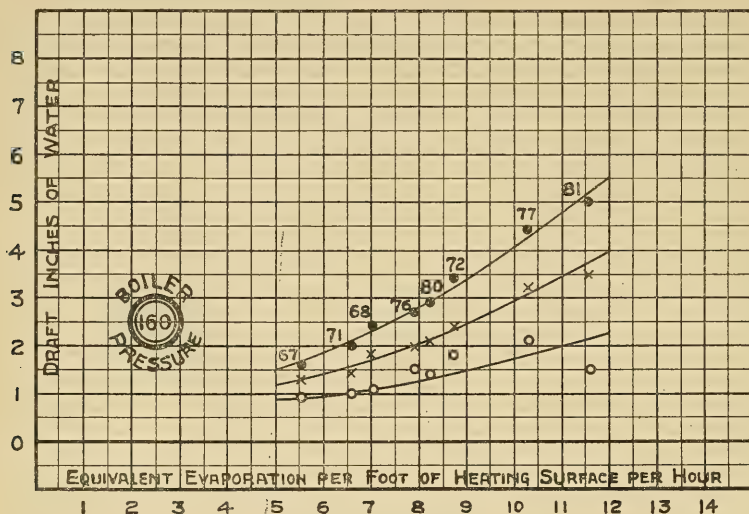


FIG. 10 DRAFT

11. *Composition of Smoke-box Gases.*—As previous experiments had shown irregularities in the evaporative efficiency of boilers of locomotives, it was early decided to proceed with care in determining the composition of the smoke-box gases. It seemed probable that if the composition of these were known for each test, variations in the evaporative efficiency of the boiler might be explained. To this end, therefore, each step in the process was carefully considered, and the work of sampling and analyzing the gases was assigned to a chemist of experience who had no other duties to perform.

The gases were drawn from the smoke-box over mercury, a period of from a half hour to an hour and a half being employed in securing the sample. The sampling-tube was of copper and of small diameter. Its length was sufficient to extend to the center of the smoke-box, and gas was admitted to it by small perforations at the extreme end only. This tube could be drawn in

and out through a stuffing-box to permit the sample to be taken either from the center of the smoke-box or from any location between that point and the shell. In securing the sample it was the practice to move the tube systematically at regular intervals of time. By these means it was assumed that abnormal results due to fluctuations in the condition of the fire would be entirely avoided.

The results, notwithstanding all precautions, have not proved entirely satisfactory; that is, where the evaporative performance is abnormal, they do not permit the assignment of a definite cause. The defects are doubtless due to faulty sampling, though it is not clear in what manner the sampling may be improved in connection with locomotive work. They do, however, entirely justify certain general conclusions. They show that the amount of excess air admitted to the furnace is never great, and in most cases it is very small—far below the limits which are thought desirable in stationary practice. They show, also, that the excess air diminishes as the rate of combustion increases. It is apparent, therefore, that the loss in efficiency arising from excess air is under normal conditions smaller than in most other classes of service. Moreover, while the supply of air appears limited, it is significant that the losses from imperfect combustion, as shown by the presence of CO, are also small, the actual amount varying irregularly between limits which are very narrow.

12. *The quality of steam* was uniformly high under all conditions for pressure, the average for all tests being 99.08. The quality declined slightly with increase of pressure, but in no case did the moisture exceed 1.35 per cent.

IV. ENGINE PERFORMANCE

13. *Mean Effective Pressure.*—A review of the calculated results shows that the possible range of cut-off under a fully-open throttle is reduced by a definite amount with each increment of pressure. For example, under 120 lb. pressure, it is possible to operate at 30 miles per hour with the reverse lever in the fourteenth notch from the center, while at 240 lb. the longest cut-off under similar conditions of speed is represented by the

fourth notch of the reverse lever. It is of interest to note, also, that within the range of the experiments each change in the position of the reverse lever results in a change in power which is nearly proportional to the extent of the movement of the reverse lever.

14. *The Indicated Horse-power.*—The range in the values of the indicated horse-power for all pressures falls between the limits of 134 and 610 horse-power. It appears from the results that with the coal used during the tests the normal power of the locomotive tested, when run at speed, is between 450 and 500 horse-power. The development of more than 500 horse-power was always attended by unusual efforts on the part of the fireman. The power of the engine, under a pressure of 240 lb., was readily developed with the reverse lever in the second and fourth notches, while under 120 lb. pressure either a high speed or a much longer cut-off must be employed before this condition is reached. All this, of course, grows out of the fact that in experiments involving a wide range of pressure the cylinder volume remained constant. It is significant that the only two tests giving a horse-power in excess of 600 lb. were run at 180 lb. and 200 lb., respectively. It will hereafter be shown that the operation of the engine under these pressures was more efficient than under conditions of pressure which were either lower or higher. Remembering that the results disclose the entire range for which it was practicable to operate the engine under a fully-open throttle, it will be accepted as a noteworthy fact that the higher pressures do not serve to increase the output of power.

15. *Steam per Indicated Horse-power per Hour.*—The high efficiency which is implied by results showing the steam consumption per indicated horse-power per hour, and the narrow range which they represent, taken in connection with the comprehensive character of the running conditions involved, are matters of more than ordinary importance. For example, at a pressure of 240 lb., the engine experimented upon, when working under a fully-open throttle, gave a horse-power hour in return for the consumption of less than 24 lb. of steam, and under any condition of speed or cut-off for which it was found possible to operate the engine under a wide-open throttle, the consumption never exceeded 26.3 lb. At lower pressures, involving the possibility of a wider choice in the conditions of operating, the range is somewhat

increased. Thus, at 120 lb. pressure the minimum value is 27.5 and the maximum 33.8, a range which, while greater than that just referred to, is nevertheless extremely narrow as compared with the range incident to the operation of other classes of engines.

The most efficient point of cut-off for the lowest pressure is that secured when the reverse lever is in the eighth notch, which is equal to 35 per cent of the stroke. At 200 lb. pressure the most efficient cut-off is that represented by the sixth notch, or 27 per cent of the stroke, and the data do not disclose that a shorter cut-off than this under a fully-open throttle is profitable for the engine experimented upon, even though the pressures be raised to 240 lb. In all cases the best results are obtained at a speed either of 20 or 40 miles an hour; for all pressures above 160 lb., the most efficient speed is 40 miles. The law of the change of efficiency with changes in speed has been discussed and the reasons underlying pointed out elsewhere.¹

The least steam consumption for each speed under the several different pressures employed is set forth in Fig. 11. The values of the figure are of interest. They do not, however, constitute a satisfactory base upon which to form comparisons.

SPEED—MILES PER HOUR	BOILER PRESSURE					
	120	160	180	200	220	240
50	28.12	26.12	24.43	25.74	24.08	24.97
40	27.51	25.82	23.68	24.43	23.68	23.86
30	27.46	25.28	24.61	24.91	23.59	24.43
20	28.40	26.14	25.44	26.01	25.51	24.09
AVERAGE	27.87	25.84	24.54	25.27	24.21	24.34

FIG. 11 LEAST STEAM FOR EACH OF THE SEVERAL SPEEDS AT DIFFERENT PRESSURES

16. *Steam Consumption under Different Pressures.*—The shaded zone upon Fig. 12 represents the range of performance as it appears from all tests run under the several pressures employed.

¹ Locomotive Performance, published by John Wiley & Sons.

For purposes of comparison, it is desirable to define the effect of pressure on performance by a line, and to this end an attempt has been made to reduce the zone of performance to a representative line. In preparing to draw such a line, the average performance of all tests at each of the different pressures was obtained and plotted, the results being shown by the circles in Fig. 12. Points thus obtained can be regarded as fairly representing the performance of the engine under the several pressures only so far as the tests run for each different pressure may be assumed to fairly represent the range of speed and cut-off under which the engine would ordinarily operate. The best result for each different pressure, as obtained by averaging the best results for each speed at constant pressure, is given upon the diagram in the form of a light cross. These points may be regarded as furnishing a satisfactory basis of comparison in so far as it may be assumed that when the speed has been determined, an engine in service will always operate under conditions of highest efficiency. Again, the left-hand edge of the shaded zone represents a comparison based on maximum performance at whatever speed or cut-off. In addition to the points already described, there is located upon the diagram (Fig. 12) a curve showing the performance of a perfect engine,¹ with which the plotted points derived from the data of tests may be compared. Guided by this curve representing the performance of a perfect engine; a line, *AB*, has been drawn proportional thereto, and so placed as to fairly represent the circular points derived from the experiments. It is proposed to accept this line as representing the steam consumption of the experimental engine under the several pressures employed. It is to be noted that it is not the minimum performance nor the maximum, but it is a close approach to that performance which is suggested by an average of all results derived from all tests which were run. Since its form is based upon a curve of perfect performance, it has a logical basis, and since it does no violence to the experimental data, its use seems justifiable.

17. *Performance under Different Pressures, A Logical Basis for Comparison.*—The record of boiler performance as set forth in

¹ This curve represents the performance of an engine working on Carnot's cycle, the initial temperature being that of steam at the several pressures stated, and the final temperature being that of steam at 1.3 lb. above atmospheric pressure. This latter value is the assumed pressure of exhaust in locomotive service.

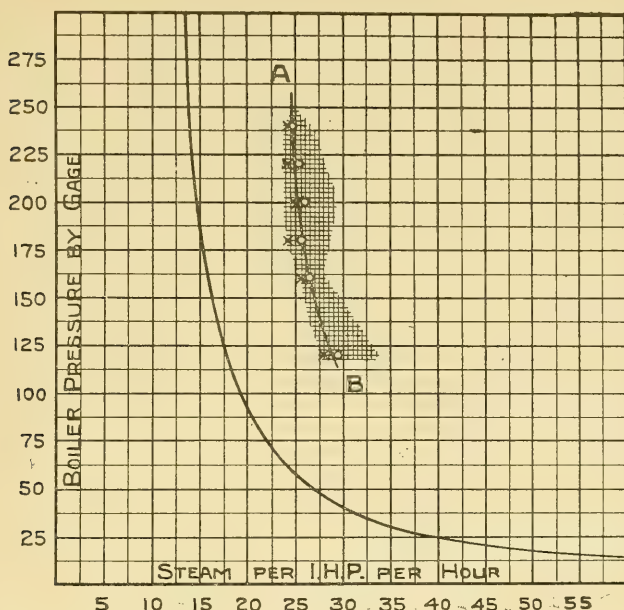


FIG. 12 STEAM CONSUMPTION UNDER DIFFERENT PRESSURES

Chapter III, is that actually obtained from the several tests run. It has already been shown that this performance is affected by variations in the evaporative efficiency of the boiler, due doubtless to irregularities in firing, but which are in fact unaccounted for. One of the purposes of the discussion which occupies the preceding chapter has been to reduce the values actually resulting from the tests to a summarized statement which may be accepted as a general definition of performance, assuming all irregularities to have been eliminated. Such a summarized statement is that which is shown by Fig. 9. It is also expressed by the equation

$$E = 11.305 - 0.221 H$$

It is now proposed to determine the coal consumption per indicated horse-power, assuming the boiler efficiency to have been in all cases that which is expressed by this equation.

It appears, also, from the data that the steam consumed by the cylinders varies for each different pressure with changes in speed and cut-off, and it has been sought in the preceding para-

graphs to summarize the facts derived from the experiments into a single expression. This appears in the form of the curve *AB*, Fig. 12, which is to be accepted as representing the performance of the cylinders under different pressures without reference to speed or cut-off. Combining this general statement expressing cylinder performance with that already obtained covering boiler performance, it should be possible to secure an accurate measure of the coal consumption per indicated horse-power hour, for each different pressure which will represent the results of all tests at that pressure.

The steps in this process are set forth by Table 2, in which—

Column 1 gives the several pressures embraced by the experiments.

Column 2 gives the steam consumption per indicated horse-power hour for each of these several pressures as taken from the curve *AB*, Fig. 12.

Column 3 gives the number of thermal units in each lb. of steam at the several pressures assuming the feed-water in all cases to have had a temperature of 60° F. The values of this column show at a glance the rate of change in the amount of heat required to supply steam at the different pressures embraced by the experiments.

Column 4 gives the pounds of water from and at 212° F. per indicated horse-power hour. It equals Column 2 $\times$ Column 3 $\div$ 965.8.

Column 5 gives the pounds of water evaporated from and at 212° F. per pound of coal and is calculated as follows: Assuming that a fair average load for the locomotive tests is 440 horse-power, and that this unit of power is delivered under all pressures, the corresponding rate of evaporation may be found by multiplying this value by those of Column 4 and dividing by the area of heating-surface; that is, the rate of evaporation = $440 \times \text{Column 4} \div 1322$. The equivalent pounds of water per pound of coal is found by substituting the rates of evaporation found for *H* in the equation,

$$E = 11.305 - 0.221 H.$$

Column 6 gives the pounds of coal per indicated horse-power per hour and equals Column 4 $\div$ Column 5.

Column 7 gives the pounds of coal saved per horse-power hour for each 20-lb. increment in steam-pressure.

Column 8 gives the percentage saving in coal for each 20-lb. increment in steam-pressure.

TABLE 2
ENGINE PERFORMANCE UNDER DIFFERENT PRESSURES

Boiler Pressure	Steam per Indicated Horse-power per Hour. Values from Curve	B.t.u. Given to 1 Pound Steam Feed-water Temp. = 60.°	Equivalent Pounds of Water per Indicated Horse-power Hour	Equivalent Pounds of Water per Pound of Dry Coal	Pounds of Coal per Indicated Horse-power Hour	Coal Saving for Each Increment	
						Lb.	Per cent
1	2	3	4	5	6	7	8
240	24.7	1176.6	30.09	9.10	3.31	.06	1.8
220	25.1	1174.4	30.52	9.06	3.37	.06	1.8
200	25.5	1172.0	30.94	9.03	3.43	.07	2.0
180	26.0	1169.5	31.48	8.99	3.50	.09	2.5
160	26.6	1166.8	32.14	8.94	3.59	.18	4.8
140	27.7	1163.8	33.38	8.85	3.77	.23	5.8
120	29.1	1160.5	34.97	8.73	4.00	...	...

The values of Table 2, especially those of Columns 2 and 6, are of more than ordinary significance. They represent logical conclusions based upon the results of all tests. Comparisons between them will show the extent to which the performance of a locomotive will be modified by changes in the steam-pressure under which it is operated. They show in the matter of steam consumption (Column 2) that—

Increasing pressure from 160 to 180 lb. reduces the steam consumption 0.6 lb. or 2.3 per cent.

Increasing pressure from 180 to 200 lb. reduces the steam consumption 0.5 lb. or 1.9 per cent.

Increasing pressure from 200 to 220 lb. reduces the steam consumption 0.4 lb. or 1.6 per cent.

Increasing pressure from 220 to 240 lb. reduces the steam consumption 0.4 lb. or 1.6 per cent.

In the matter of coal consumption (Column 6) they show that—

Increasing pressure from 160 to 180 lb. reduces the coal consumption 0.9 lb. or 2.5 per cent.

Increasing pressure from 180 to 200 lb. reduces the coal consumption 0.7 lb. or 2.0 per cent.

Increasing pressure from 200 to 220 lb. reduces the coal consumption 0.6 lb. or 1.8 per cent.

Increasing pressure from 220 to 240 lb. reduces the coal consumption 0.6 lb. or 1.8 per cent.

These values are from actual tests. Those who are inclined to insist upon basing their conclusions upon observed data will perhaps find in them a satisfactory conclusion of the whole investigation. The results show how slight is the gain to be derived from any increment of pressure when the basis of the increments is above 160 lb. But they do not in fact tell the whole story. In order to secure such results from a single locomotive it was necessary to employ a machine designed for the highest pressure experimented upon. Obviously, for the tests at lower pressure, the locomotive was needlessly heavy for its dimensions. If, for the tests under each of the lower pressures, the excess weight could have been utilized in providing a boiler of greater heating-surface, the difference in performance with each increment of pressure would have been less than that to which attention has already been called. It is for this reason that the results already quoted, while significant and concise in their meaning, are nevertheless to be accepted as insufficient when regarded as a relative measure of the value of different steam-pressures. An extension of the discussion leading to a more general view of the matter will be found set forth in Chapters VI to VIII.

V. MACHINE FRICTION AND PERFORMANCE AT DRAW-BAR

18. *The Cylinders vs. the Draw-Bar as a Base from Which to Estimate Performance.*—In the latter paragraphs of the preceding chapter results are given disclosing the performance of boiler and engine as based upon cylinder performance. This is a correct basis from which to proceed in discussing the relative advantage of different steam-pressures; for the process of the cylinders represents the last of the thermodynamic changes by which the heat of the fuel is transformed into work. The cylinders are in fact one step nearer the problem in question than the draw-bar, which for many purposes is properly regarded as a better basis from which to determine the performance of a locomotive. This being the case, the purpose of the present chapter will be entirely served if attention is called to a few of the more significant facts which center in the output of power at the draw-bar, leaving the

general discussion as to the relative values of different steam-pressures to be continued in the chapters which follow.

19. *Machine Friction.*—This is the difference between work done in the engine cylinders and that which appears at the draw-bar. It is difficult to summarize the facts concerning engine friction. This is not due to defects in the experimental process underlying the data, but to the fact that the frictional resistance of the machinery of the locomotive varies greatly from day to day.¹ Evidence of this is accessible even to the casual observer. During any given test it is likely that an axle-box or a crank-pin may run warm, while during another test under identical conditions of power the same part will remain perfectly cool. For this reason many variations in the frictional resistance of the machine occur. It is a fact, however, that the friction varies but slightly with increase in steam-pressure, and that changes in cut-off are most effective in modifying engine friction. Acting upon this suggestion, all results have been plotted in terms of cut-off. The results do not, of course, fall in line, but they take such positions as readily to suggest the form of a curve which in an approximate way may be employed to represent them. From such a curve the values set forth in Fig. 13 have been derived. It is proposed to accept

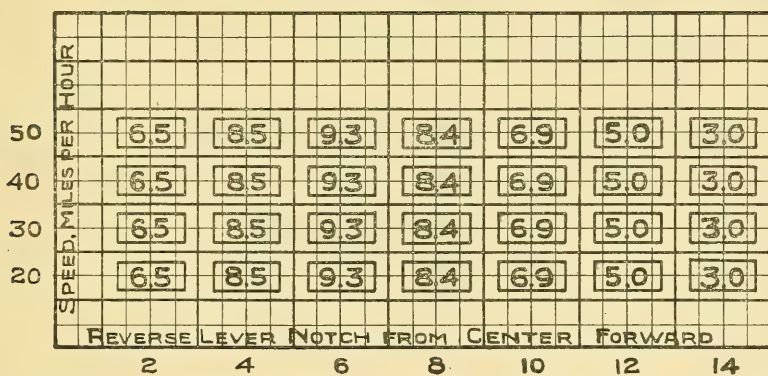


FIG. 13 CORRECTED FRICTION, MEAN EFFECTIVE PRESSURE
APPLICABLE TO ALL PRESSURES

these values as an approximate measure of the frictional loss for locomotive *Schenectady No. 2* under all pressures. They are probably a little low for pressures above 200 lb. and are perhaps

¹ A general discussion of this question with data will be found in *Locomotive Performance*.

somewhat high for pressures below this limit. It can not be assumed that they apply to any other locomotive than that which was involved by the experiments. The machine friction as expressed in pounds pull at the draw-bar may be found for any test by multiplying the mean effective pressure for that test by the constant 88.75.

20. *Steam per Dynamometer Horse-power per Hour.*—Values covering this factor express the combined efficiency of the cylinders and machinery of the locomotive. They disclose the fact that there are few conditions of running for which the locomotive requires more than 30 lb. of steam per dynamometer horse-power hour, and the consumption may fall below 27 lb. While differences in performance for all pressures above 200 lb. are not great, the steam consumption is much greater when the pressure is as low as 120 lb. The data show, also, that for best results the cut-off must be lengthened as the pressure is decreased. The facts as disclosed by the data are as follows:

For 240 lb. pressure the best cut-off is approximately the second notch, 14 per cent.

For 220 lb. pressure the best cut-off is approximately the fourth notch, 19 per cent.

For 180 lb. pressure the best cut-off is approximately the eighth notch, 33 per cent.

For 120 lb. pressure the best cut-off is approximately the twelfth or fourteenth notch, 47 per cent or 56 per cent.

21. *Coal per Dynamometer Horse-power per Hour.*—This factor represents the combined performance of the boiler, the cylinders, and the machinery of a locomotive. It connects the energy developed in the boiler by the combustion of fuel with that which is developed at the draw-bar. In all cases where data are given, the fuel consumed was of the same quality; hence all results are comparable. Under a pressure of 240 lb. the range is between 3.35 and 5.01, while at a pressure of 160 lb. the range is between 3.79 and 4.78, results which are of interest from at least two points of view: first, because of the small difference in performances resulting from a relatively large change in pressure; and second, because of the significance of the values quoted when accepted as a measure of the locomotive performance. It is doubtful if any other type of steam-engine exhausting into the atmosphere can

be depended upon to deliver power from the periphery of its wheel in return for the expenditure of so small an amount of fuel.

22. *Corrected Results.*—The values representing coal and steam consumption which have thus far been referred to as performance at the draw-bar are those actually observed. A close comparison of these sometimes fails to give consistent results because of irregularities in boiler performance or in the frictional resistance of the machinery growing out of causes already discussed.

In Table 3 values are presented from which all such discrepancies have been eliminated. They are those which would have been obtained if the evaporative efficiency for all tests had been that indicated by the equation:

$$E = 11.305 - 0.221 H$$

and if the machine friction for all cases had been that shown by Fig. 13. Column 13 giving the corrected coal per dynamometer horse-power, and Column 14 the corrected steam per dynamometer horse-power, may be accepted as representing the best information derived from the entire research.

TABLE 3

COMPARATIVE PERFORMANCE OF THE LOCOMOTIVE, ASSUMING IRREGULARITIES IN THE RESULTS OF INDIVIDUAL TESTS TO HAVE BEEN ELIMINATED

Number	Designation of Tests	Equivalent Steam to Engine per Hour. Feed-water at 60° F.	Equiv. Evap. per Pound of Dry Coal by Equation $F = 11.305 - .221 H$	Dry Coal Fired per Hour Corrected by Equation, pounds	Dry Coal per I. H. P. per Hour, pounds	Equiv. Steam per I. H. P. per Hour, pounds	Machine Friction			Dynamometer Horse-power	Draw-bar Pull pounds	Coal per D. H. P. per Hour, pounds	Equiv. Steam per D. H. P. per Hour pounds
							M. E. P.	H. P.	Per cent I. H. P.				
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	20-2-240	8803	9.835	895	3.24	31.84	6.5	30.8	11.1	245.6	4600	3.64	35.86
2	20-4-240	12008	9.298	1291	3.29	30.59	8.5	40.2	10.2	352.3	6610	3.66	34.08
3	20-6-240	13614	9.029	1508	3.23	29.12	9.3	44.0	9.4	422.8	7930	3.56	32.20
5	30-2-240	11444	9.392	1218	3.28	30.82	6.5	46.1	12.4	325.2	4060	3.74	35.19
6	30-4-240	13888	8.983	1546	3.28	29.51	8.5	60.4	12.8	410.2	5127	3.77	33.85
8	40-2-240	12320	9.245	1333	3.16	29.20	6.5	61.5	14.6	360.3	3379	3.69	34.19
9	40-4-240	16320	8.576	1903	3.36	28.82	8.5	80.5	14.2	485.8	4550	3.91	33.59
11	50-2-240	14066	8.953	1571	3.37	30.21	6.5	76.9	16.5	388.6	2910	4.04	36.19
13	20-2-220	8533	9.878	864	3.38	33.42	6.5	30.8	12.0	224.5	4210	3.84	38.01
14	20-4-220	10681	9.519	1122	3.27	31.15	8.5	40.2	11.7	302.6	5670	3.71	35.29
15	20-6-220	13294	9.082	1463	3.39	30.81	9.3	44.0	10.2	387.4	7260	3.77	34.31
16	20-8-220	16653	8.521	1954	3.66	31.24	8.4	39.8	7.5	493.2	9250	3.96	33.76
17	30-2-220	10286	9.585	1073	3.35	32.11	6.5	46.1	14.4	274.2	3430	3.91	37.51
18	30-4-220	12976	9.136	1420	3.20	29.06	8.5	60.4	13.5	386.1	4820	3.68	33.60
19	30-6-220	15915	8.644	1841	3.29	28.44	9.3	66.0	11.8	493.5	6170	3.73	32.25
21	40-2-220	11471	9.387	1222	3.29	30.87	6.5	61.5	16.5	310.0	2910	3.94	37.00
22	40-4-220	14549	8.873	1638	3.21	28.57	8.5	80.5	15.8	428.6	4020	3.82	33.94
24	50-2-220	12017	9.296	1292	3.41	31.72	6.5	76.9	20.3	301.9	2260	4.28	39.80
25	50-4-220	16343	8.573	1906	3.39	29.08	8.5	100.6	17.9	461.7	3460	4.13	35.40
29	20-2-200	7632	10.029	761	3.40	34.14	6.5	30.8	13.8	192.7	3610	3.94	39.61
30	20-4-200	9100	9.784	930	3.23	31.64	8.5	40.2	14.0	247.4	4640	3.75	36.78
31	20-6-200	11774	9.337	1261	3.35	31.33	9.3	44.0	11.7	331.8	6220	3.80	35.48
32	20-8-200	15011	8.795	1707	3.60	31.74	8.4	39.8	8.4	433.1	8120	3.94	34.66
33	30-2-200	8768	9.839	891	3.31	32.60	6.5	46.1	17.1	222.8	2780	4.00	39.35
34	30-4-200	11354	9.406	1207	3.29	30.92	8.5	60.4	16.4	306.7	3830	3.93	37.02
35	30-6-200	14685	8.850	1659	3.39	30.00	9.3	66.0	13.5	423.2	5290	3.92	34.70
37	40-2-200	9934	9.644	1030	3.36	32.40	6.5	61.5	20.0	245.1	3300	4.22	40.53
38	40-4-200	13361	9.071	1473	3.28	29.76	9.5	80.5	17.9	368.4	3450	4.00	36.27
39	40-6-200	17822	8.321	2142	3.54	29.54	9.3	88.0	14.5	517.2	4850	4.14	34.46
41	50-2-200	10206	9.599	1074	3.26	31.02	6.5	76.9	23.4	252.1	1890	4.26	40.48
42	50-4-200	14431	8.892	1623	3.49	31.08	8.5	100.6	1.6	363.6	2730	4.46	39.69
46	20-2-180	6638	10.195	651	3.40	34.57	6.5	30.8	16.0	161.2	3020	4.04	41.18
47	20-4-180	8475	9.888	858	3.25	32.15	8.5	40.2	15.3	223.4	4190	3.84	37.94

TABLE 3 (Continued)

Number	Description of Tests	Equivalent Steam to Engine per Hour. Feed-water at 60° F.	Equiv. Evap. per Pound of Dry Coal by Equation $E = 11.305 - .321 H$.	Dry Coal Fired per Hour Corrected by Equation, pounds	Dry Coal per I. H. P. per Hour, pounds	Equiv. Steam per I. H. P. per Hour, pounds	Machine Friction			Dynamometer Horse-power	Draw-bar Pull, pounds	Coal per D. H. P. per Hour, pounds	Equiv. Steam per D. H. P. per Hour, pounds
							M. E. P.	H. P.	Per cent I. H. P.				
1	2	3	4	5	6	7	8	9	10	11	12	13	14
48	20-6-180	10226	9.595	1066	3.19	30.61	9.3	44.0	13.2	290.1	5440	3.67	35.25
49	20-8-180	12833	9.157	1401	3.40	31.17	8.4	39.8	9.7	371.9	6970	3.77	34.51
51	30-2-180	7523	10.047	749	3.18	31.91	6.5	46.1	19.5	189.6	2370	3.95	39.68
52	30-4-180	9722	9.680	1004	3.16	30.65	8.5	60.4	19.1	256.7	3210	3.91	37.87
53	30-6-180	11633	9.360	1243	3.16	29.58	9.3	66.0	16.8	327.3	4090	3.80	35.54
54	30-8-180	16156	8.604	1878	3.44	29.57	8.4	59.6	10.9	486.7	6080	3.86	33.20
56	40-2-180	8069	9.956	810	3.12	31.14	6.5	161.5	23.7	197.6	1850	4.10	40.84
57	40-4-180	11177	9.436	1184	3.07	28.94	8.5	80.5	20.8	305.7	2870	3.87	36.56
58	40-6-180	14907	8.813	1691	3.23	28.44	9.3	88.0	16.8	436.1	4090	3.88	34.18
59	40-8-180	18949	8.137	2329	3.82	31.07	8.4	79.5	13.0	530.4	4970	4.39	35.73
61	50-2-180	8578	9.871	869	3.24	32.01	6.5	76.9	28.7	191.1	1430	4.55	44.88
62	50-4-180	12061	9.288	1299	3.16	29.37	8.5	100.6	24.5	310.0	2320	4.19	38.90
63	50-6-180	16567	8.535	1941	3.51	29.94	9.3	110.1	19.9	443.2	2320	4.38	37.40
67	20-4-160	7396	10.068	734	3.34	33.69	8.5	40.2	18.4	179.3	3360	4.09	41.25
68	20-6-160	9379	9.737	963	3.27	31.87	9.3	44.0	14.9	250.4	4690	3.85	37.44
69	20-8-160	11392	9.400	1212	3.51	33.02	8.4	39.8	11.5	305.2	5720	3.97	37.33
71	30-4-160	8785	9.836	893	3.28	32.28	8.5	60.4	22.2	211.7	2640	4.22	41.50
72	30-6-160	11663	9.355	1246	3.25	30.38	9.3	66.0	17.2	317.9	3970	3.92	36.69
73	30-8-160	14347	8.906	1611	3.46	30.85	8.4	59.6	12.8	405.4	5070	3.97	35.39
76	40-4-160	10106	9.615	1051	3.31	31.83	8.5	80.5	25.4	237.0	2220	4.43	42.64
77	40-6-160	13406	9.065	1478	3.43	31.05	9.3	88.0	20.4	343.7	3220	4.30	39.00
78	40-8-160	17246	8.421	2048	3.76	31.70	8.4	79.5	14.6	464.4	4350	4.41	37.14
80	50-4-160	10982	9.469	1160	3.43	32.47	8.5	100.6	29.7	237.7	1773	4.89	46.20
81	50-6-160	14940	8.807	1696	3.56	31.39	9.3	110.1	23.1	365.8	2740	4.64	40.84
85	20-4-120	5215	10.433	500	3.73	38.92	8.5	40.2	30.0	93.8	1760	5.33	55.59
86	20-8-120	8592	9.869	871	3.44	33.99	8.4	39.8	15.7	213.0	3990	4.09	40.34
87	20-12-120	12329	9.244	1333	3.73	34.52	5.0	23.7	6.5	333.2	6250	4.00	37.00
88	30-4-120	6269	10.257	611	3.57	36.69	8.5	60.4	35.4	110.6	1380	5.52	56.68
89	30-8-120	10683	9.519	1122	3.45	32.80	8.4	59.6	18.3	265.9	3320	4.22	40.18
90	30-14-120	18654	8.186	2278	4.43	36.29	3.0	21.3	4.1	492.7	6160	4.62	37.86
91	40-4-120	6649	10.193	652	3.54	36.13	8.5	80.5	43.7	103.5	970	6.30	64.24
92	40-8-120	12796	9.166	1396	3.59	32.89	8.4	79.5	20.4	309.5	2900	4.51	41.34
93	40-12-120	18942	8.138	2328	4.20	34.12	5.0	47.3	8.5	507.5	4760	4.58	37.32
94	50-4-120	7129	10.113	704	4.00	40.51	8.5	100.6	57.2	75.4	560	9.34	94.55
95	50-8-120	14371	8.902	1614	3.77	33.61	8.4	99.4	23.2	328.2	2460	4.91	43.79
96	50-11-120	19317	8.075	2391	4.32	34.90	6.0	71.0	12.8	482.5	3620	4.95	40.04

VI. BOILER PRESSURE AS A FACTOR IN ECONOMICAL OPERATION

23. *The amount of steam consumed by the locomotive per unit power developed, when operated under various pressures between the limits of 120 lb. and 240 lb., has already been defined (Fig. 12). Basing conclusions on results thus disclosed, it is now proposed to determine the increase in efficiency which may be secured through the adoption of higher pressure for any given increase in the weight of the boiler and its related parts. That this may be done, it is essential to determine the relation between boilers of a given size when designed for different pressures.*

24. *Weight of Locomotive as Affected by Steam-Pressure.*—The parts of a locomotive which are affected by changes in steam-pressure, assuming the power to remain constant, are the boiler and certain portions of the engine. The boiler to be adapted to a higher steam-pressure requires thicker plates, heavier riveting, and stronger staying, all tending to augment its weight. The effect of the change upon the engine, however, is to make it lighter, for since with increased pressure, cylinders, pistons, and valves become smaller, their weight will generally diminish. As a basis for exact values, defining their relationship, lines were laid down for a boiler of the following dimensions:¹

Diameter of first ring.....	inches	63
Number of 2-inch tubes.....		258
Length of tubes ..	feet	14
Total heating-surface.....	square feet	2024
Length of grate.....	inches	90
Width of grate.....	inches	60
Area of grate.....	feet	37.5
Boiler-pressure	pounds	190

Four designs were made, adapted to four different pressures, respectively, from which designs weights were calculated, with results shown by Table 4.

The weight of the cylinders, valves, and pistons which would be used with a boiler having 2024 sq. ft. of heating-surface in making up a representative locomotive carrying the different pressures designated is set forth in Column 3. The weight of water when the boiler is filled to the second gage appears as Column 4. The weight of steam is negligible. The total weight of all parts

¹ These and other determinations involve weights of boilers which were supplied by the courtesy of the American Locomotive Company.

TABLE 4

WEIGHT OF THOSE PARTS OF A LOCOMOTIVE WHICH ARE AFFECTED BY CHANGES IN BOILER PRESSURE

Boiler Pressure	Weight of Boiler pounds	Weight of Cylinders, Valves, and Pistons pounds	Weight of Water pounds	Weight of All Parts Affected by Changes in Pressure pounds
1	2	3	4	5
160	30679	12580	16349	59608
190	32913	12240	16536	61689
220	36076	11990	16661	64727
250	38953	11620	16848	67421

of the locomotive directly affected by the changes in pressure is given in Column 5, and the values of this column, for the purpose of interpolation, have been plotted in terms of steam-pressure, with results as set forth by Fig. 14.

With these data it is proposed to show the extent to which the performance of a typical locomotive using saturated steam may be improved by increasing the pressure carried within its boiler. For convenience, six different pressures having values between 120 lb. and 220 lb. will be utilized as bases from which to

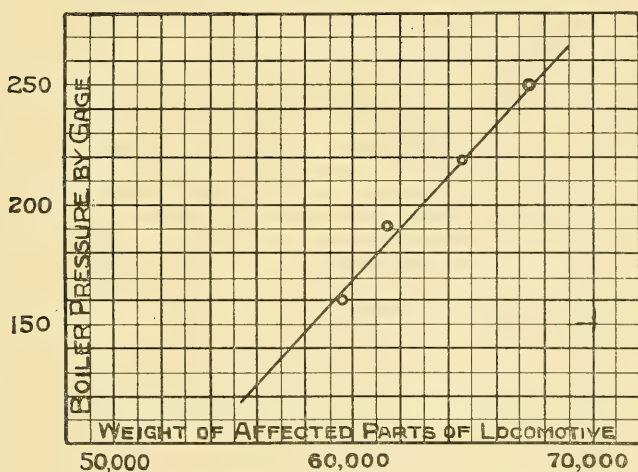


FIG. 14 WEIGHT OF BOILER AS AFFECTED BY CHANGES IN PRESSURE

assume an increase of pressure. The increase of pressure from each base will be such as may be possible upon the allowance of definite increments in the weight of those portions of the locomotive affected by pressure, and in like manner the improvement in performance will be expressed as a per cent of that which is normal to the base. The results of the process outlined are presented in Table 5. An explanation of the columns of this table whose meaning is not self-evident follows:

Column 3. Weight of those parts of a typical locomotive affected by changes in steam-pressure, including water in boiler.—The values of this column, for each of the several pressures stated in Column 2, are taken directly from the diagram of Fig. 14, the basis of which has already been explained.

Column 5. New boiler-pressure obtainable by utilizing the increase of weight in making a stronger boiler.—The values in this column for each of the several weights stated in Column 4 were taken from the diagram in Fig. 14.

Column 6. Steam per indicated horse-power per hour at the pressures given in Column 2.—Values for this column are taken directly from the curve of Fig. 12.

Column 7. Steam per indicated horse-power per hour at the new pressures given in Column 5.—These values, also, were taken directly from the diagram (Fig. 12).

Column 8. Direct saving in steam consumption, resulting from an increased weight equal to the per cent shown in Column 1.—Values of this column are equal to 100 times those of Column 6 minus those of Column 7 divided by those of Column 6.

Column 9. Indirect saving due to reduced rates of evaporation, per cent.—Assuming the locomotive to work at the same power at whatever pressure it may carry, the saving in steam resulting from the increased pressure set forth in Column 8 diminishes the demand upon the boiler, and, as the efficiency of the boiler increases as the rate of evaporation is reduced, there results an indirect saving with each increase of pressure. The relation between the evaporative efficiency of the boiler and rate of evaporation has already been defined (Fig. 9). Assuming the normal rate of evaporation for the boiler under initial conditions to be 10, then a reduction of 1 per cent in the rate of evaporation will effect an increase in the evaporative efficiency of 0.243 per cent. The values in Column 9, therefore, are those of Column 8 multiplied by the constant 0.243.

TABLE 5

TOTAL SAVING WHEN A POSSIBLE INCREASE OF WEIGHT IS UTILIZED AS
A MEANS OF INCREASING BOILER-PRESSURE

Increase of Weight per cent	Boiler-pressures Selected as Bases pounds	Weight of Those Parts of a Locomotive Which Are Affected by Changes in Boiler-pressure pounds	Weight of Affected Parts Increased by per cent Given in Column 1 pounds	New Boiler-pressure Obtainable by Utilizing the Increase of Weight in Making a Stronger Boiler, pounds	Steam per Indicated Horse-power per Hour at the Pressures Given in Column 2, pounds	Steam per Indicated Horse-power per Hour at the New Pressures Given in Column 5, pounds	Direct Saving in Steam Consumption Resulting from an Increased Weight Equal to per cent Shown in Column 1, per cent	Indirect, Saving Due to Reduced Rate of Evaporation, per cent	Total Saving per cent
1	2	3	4	5	6	7	8	9	10
5	120	55560	58340	150	29.1	27.1	6.87	1.67	8.54
	140	57390	60260	171	27.7	26.3	5.05	1.23	6.28
	160	59220	62180	192	26.6	25.7	3.39	.82	4.21
	180	61050	64100	213	26.0	25.2	3.08	.75	3.83
	200	62880	66020	234	25.5	24.8	2.75	.67	3.42
10	220	64710	67940	255	25.1	24.5	2.39	.58	2.97
	120	55560	61120	181	29.1	26.0	10.65	2.59	13.24
	140	57390	63130	203	27.7	25.4	8.31	2.02	10.33
	160	59220	65140	225	26.6	25.0	6.02	1.46	7.48
	180	61050	67150	247	26.0	24.6	5.38	1.31	6.69
15	120	55560	63890	211	29.1	25.3	13.06	3.17	16.23
	140	57390	66000	234	27.7	24.8	10.46	2.51	13.00
20	160	59220	68100	257	26.6	24.5	7.90	1.92	9.82
	120	55560	66670	241	29.1	24.7	15.12	3.67	18.79

Column 10. *Total saving.*—The total saving is the sum of Columns 8 and 9.

The significance of this table may best be appreciated by the following examples:

By line 1 of the table it appears that the base is 120 lb. (Column 2). The parts of the typical locomotive designed for this pressure, which are affected by changes in steam-pressure, weigh 55,560 lb. (Column 3). If, now, in designing a new lot of locomotives, it becomes possible to increase this weight by 5 per cent (Column 1), the weight of these parts for the new locomotive may be 58,340 lb. (Column 4). This weight, if put into a boiler of the same capacity, will allow the pressure to be increased from 120 lb. (Column 2) to 150 lb. (Column 5), and as a result its steam con-

sumption per horse-power hour will fall from 29.1 lb. (Column 6) to 27.1 lb. (Column 7), or 6.87 per cent (Column 8). But the saving of 6.87 per cent in steam consumption diminishes the demand which is made upon the boiler for steam, and at the lower rate of evaporation the boiler becomes 1.67 per cent (Column 9) more efficient, giving a total gain as a result of the change in pressure of 8.58 per cent (Column 10). In a similar manner each line of the table presents a measure of the improvement to be expected from some definite increase of pressure.

A study of the analysis which has preceded will show that the values of Column 10 may be accepted as fairly representing the increase in efficiency which may be secured in return for a given increase in steam-pressure, or, as is more clearly shown by Table 4, in return for a given increase in the weight of those parts of the locomotive affected by increase of pressure.

While the comparison is based on improved efficiency, it will, of course, be understood that, at the limit, the saving shown may be converted into a corresponding increase of power. It would have been possible by assuming constant efficiency to have shown the improvement in terms of increase of power.

VII. BOILER CAPACITY AS A FACTOR IN ECONOMICAL OPERATIONS

25. *In the preceding chapter there is considered the advantage to be derived through the utilization of any possible increase in the weight of a locomotive, as a means by which to secure an increase of pressure. It is the purpose of this chapter to consider the benefit which may be derived by utilizing similar increments in weight to secure an increase in boiler capacity, the pressure remaining constant. The weights of boilers and related parts involved by such a comparison have been ascertained from considerations similar to those which controlled in the preceding case. A boiler of the dimensions already given (paragraph 24), designed for 190 lb., was made the starting-point from which values were ascertained for boilers of different capacities designed to carry 160 lb. pressure. The characteristics of the several boilers thus designed are set forth in Table 6.*

TABLE 6

CHARACTERISTICS OF FOUR BOILERS DESIGNED FOR 160 POUNDS
PRESSURE AND DIFFERENT CAPACITIES

Diam- eter of Boiler inches	Num- ber of 2-inch Tubes	Length of Tubes feet	Length of Grate inches	Width of Grate inches	Area of Grate sq. ft.	Area of Heating- surface sq. ft.	Weight of Boiler pounds	Weight of Water in Boiler pounds	Weight of Parts of Locomotive Which Are Affected by Changes in Heating- surface pounds
1	2	3	4	5	6	7	8	9	10
63	258	14	90	60	37.4	2024	30,679	16,349	47,028
67	338	16	102	65	46.1	3013	41,013	20,092	61,105
69	326	14	102	65	46.1	2538	36,321	19,344	55,665
70	396	16	96	75	50.0	3498	42,894	21,965	64,859

The steam-pressure being constant, the dimensions and consequently the weight of the cylinders and related parts for the development of a given power remain unchanged. It is obvious, also, that since the only change in the locomotive is in the size of its boiler, the cylinder performance will be the same for locomotives having boilers of different sizes. The saving which will result from the employment of boilers of greater capacity will be only that which results from the diminished rate of evaporation per unit area of heating-surface. The relation of evaporative efficiency and rate of evaporation has already been defined (Fig. 9), so that both factors in the problem now are known, namely, the increase in weight necessary for a given increase in capacity and the effect of any increase in capacity in improving the evaporative efficiency. By means of relations thus established values have been determined which are presented in Table 7. An explanation of the columns of this table whose meaning is not self-evident is as follows:

Column 3 is the weight of boiler, the contained water, and the cylinders, pistons, and valves. While the cylinders, pistons, and valves do not change for any given pressure, their weights are included to make the values comparable with those employed in the analysis of the preceding chapter. They are in fact identical with the values of Column 3, Table 5.

Column 4. Allowable increase in weight.—The values of this column are the percentages indicated by Column 1 of the values of Column 3.

TABLE 7

SAVING WHEN A POSSIBLE INCREASE OF WEIGHT IS UTILIZED
AS A MEANS OF INCREASING HEATING-SURFACE

Increase of Weight per cent	Boiler-pressures Selected as Bases pounds	Weight of Parts of a Typical Locomotive (Boiler, Cylinders, Valves, Pistons, and Water) pounds	Allowable Increase of Weight pounds	Heating-surface of Typical Locomotives Whose Weights Are Given in Column 3 sq. ft.	Increase of Heating-surface Obtainable by Utilizing Increase of Weight in Making a Larger Boiler sq. ft.	Increase of Heating-Surface per cent	Saving in Evaporative Performance Due to Reduced Rate per cent
1	2	3	4	5	6	7	8
5	120	55560	2778	2000	234.7	11.73	2.85
	140	57390	2869	2000	242.5	12.12	2.95
	160	59220	2961	2000	250.1	12.50	3.04
	180	61050	3052	2000	257.7	12.88	3.13
	200	62880	3144	2000	265.3	13.26	3.22
10	220	64710	3235	2000	272.9	13.64	3.31
	120	55560	5556	2000	469.4	23.47	5.70
	140	57390	5739	2000	484.9	24.24	5.89
	160	59220	5922	2000	500.4	25.02	6.08
	180	61050	6105	2000	515.9	25.79	6.27
15	120	55560	8334	2000	704.2	35.21	8.55
	140	57390	8608	2000	727.3	36.36	8.84
20	160	59220	8883	2000	750.6	37.53	9.12
	120	55560	11112	2000	939.0	46.95	11.41

Column 6. Increase of heating-surface.—Values for this column have been obtained by plotting weight of affected parts in terms of heating-surface (Columns 7 and 10, Table 5). The results appear in Fig. 15. From a representative line drawn through points thus obtained showing the relation between the weight of the boiler and water, and the number of square feet of heating-surface, it can be shown that an increase of 10,000 lb. in the weight of boiler and affected parts permits an increase of 845 sq. ft. in heating-surface. Therefore, in Table 6, Column 6 equals Column 4 multiplied by 0.0845. This relation was obtained from data of a boiler designed for 160 lb. pressure and is assumed to be approximately true for boilers of other pressures.

Column 7. Increase of heating-surface, per cent, is Column 6 multiplied by 100 divided by Column 5. It also shows the per cent reduction in the rate of evaporation.

Column 8. Saving in evaporative performance due to reduced rate, per cent.—Values in this column have been obtained from

those of the preceding column by means of a relationship already established controlling evaporative efficiency of boiler and rate of combustion (Fig. 9). This relation is such that a reduction of 1 per cent in the rate of combustion increases the evaporative efficiency 0.243 per cent. Values of Column 8 are, therefore, those of Column 7 multiplied by this factor.

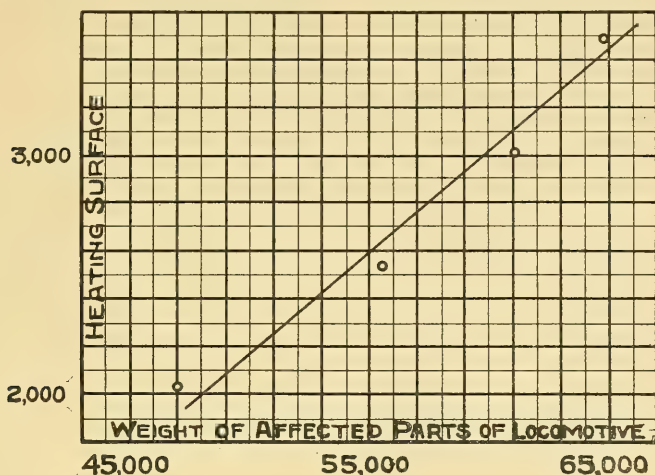


FIG. 15 WEIGHT OF BOILER AS AFFECTED BY CHANGES IN HEATING-SURFACE

The significance of Table 6 will be understood from the following illustration, based upon the first line of the table. Assuming an existing locomotive operating under a pressure of 120 lb. (Column 2) to have a boiler containing 2000 sq. ft. of heating-surface (Column 5) weighing with the contained water 55,560 lb. (Column 3), an increase of .5 per cent (Column 1) or 2778 lb. (Column 4), will permit an extension in heating-surface of 234.7 sq. ft. (Column 6) which, compared with its original surface is an increase of 11.73 per cent (Column 7). This increase in the extent of heating-surface, assuming the power developed to remain unchanged, will result in an improvement in the performance of the boiler of 2.86 per cent (Column 8). The facts underlying the analysis are primarily the results of tests.

VIII. CONCLUSIONS CONCERNING BOILER-PRESSURE *vs.* BOILER CAPACITY AS A MEANS OF INCREASING THE EFFICIENCY OF A SINGLE-EXPANSION LOCOMOTIVE

26. *In the preceding chapters an analysis has been given showing the saving which may result in locomotive service, first, by increasing the pressure, the boiler capacity remaining unchanged, and second, by increasing the heating-surface, the pressure remaining unchanged. A summary of the conclusions of these chapters is presented in Fig. 16 to 21 in which the full line represents the gain through increase of boiler-pressure and the dotted line the corresponding gain through increase of boiler capacity. The values for these diagrams are taken directly from Tables 5 and 7. It will be seen that starting with pressures which are comparatively low, the most pronounced results are those to be derived from increments of pressure. With each rise in pressure, however, the chance for gain through further increase diminishes. With a starting-point as high as 180 lb., the saving through increased pressure is but slightly greater than that which may result through increased boiler capacity.*

The fact should be emphasized that the conclusions above described are based upon data which lead back to the question of coal consumption. The gains which are referred to are measured in terms of coal which may be saved in the development of a given amount of power. It will be remembered that conditions which permit a saving in coal will, by the sacrifice of such saving, open the way for the development of greater power, but the question as defined is one concerning economy in the use of fuel. It is this question only with which the diagrams (Fig. 16 to 21) deal.

There are other measures which may be applied to the performance of a locomotive which, if employed in the present case, would show some difference in the real values of the two curves (Fig. 16 to 21). The indefinite character of these measures prevents their being directly applied as corrections to the results already deduced, but their effect may be pointed out. Thus, the extent to which an increase of pressure will improve performance has been defined, but the definition assumes freedom from leakage. If, therefore, leakage is allowed to exist, the result defined is not secured. Moreover, an increase of pressure increases the chance

of loss through leakage, so that to secure the advantage which has been defined, there must be some increase in the amount of attention bestowed, and this, in whatever form it may appear, means expense, the effect of which is to reduce the net gain which it is possible to derive through increase of pressure. Again, in parts of the country where the water-supply is bad, any increase of pressure will involve increased expense in the more careful and more extensive treatment of feed-water, or in the increased cost of boiler repairs, or in detentions arising from failure of injector, or from all of these sources combined. The effect of such expense is to reduce the net gain which it is possible to derive through increase of pressure. These statements call attention to the fact that the gains which have been defined as resulting from increase of pressure (Fig. 16 to 21) are to be regarded as the maximum gross; as maximum because they are based upon results derived from a locomotive which was at all times maintained in the highest possible condition, and as gross because on the road, conditions are likely to be introduced which will necessitate deductions therefrom.

The relation which has been established showing the gain to be derived through increased boiler capacity is subject to but few qualifying conditions. It rests upon the fact that for the development of a given power a large boiler will work at a lower rate of evaporation per unit area of heating-surface than a smaller one. The saving which results from diminishing the rate of evaporation is sure; whether the boiler is clean or foul, tight or leaky, or whether the feed-water is good or bad, the reduced rate of evaporation will bring its sure return in the form of increased efficiency. An increase in the size of a boiler will involve some increase in the cost of maintenance, but such increase is slight and of a sort which has not been regarded in the discussion involving boilers designed for higher pressures.

Keeping in mind the fact that as applied to conditions of service the line *A* is likely to be less stable in its position than *B*, the facts set forth by Fig. 16 to 21 may be briefly reviewed.

Basing comparisons upon an initial pressure of 120 lb., (Fig. 16), a 5 per cent increase in weight, when utilized in securing a stronger boiler, will improve the efficiency 8.5 per cent, while if utilized in securing a larger boiler, the improvement will be a trifle less than 3 per cent. Arguing from this base, the advantage

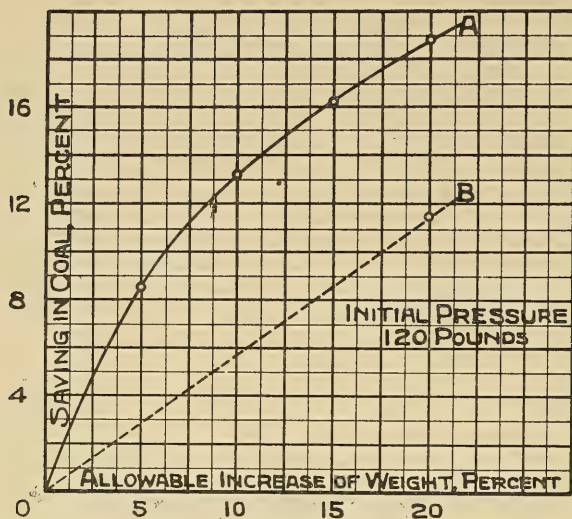


FIG. 16

The line A represents the saving in fuel when an allowable increase in weight is utilized in making a stronger boiler to permit a higher pressure.

The line B represents the saving in fuel when an allowable increase in weight is utilized in making a larger boiler to give increased capacity.

to be derived from an increase of pressure is great. If, however, the increase in weight exceeds 10 per cent, the curve A ceases to diverge from B and if both curves are sufficiently extended, they will meet, all of which is proof of the fact that the rate of gain is greatest for relatively small increments of weight.

Basing comparisons upon an initial pressure of 140 lb. (Fig. 17), the relative advantage of increasing the pressure diminishes, though on the basis of a 5 per cent increase in weight it is still double that to be obtained by increasing the capacity.

Basing comparisons upon an initial pressure of 160 lb. (Fig. 18), the advantage to be gained by increasing the pressure over that which may be had by increasing the capacity is very small, so small in fact that a slight droop in the curve of increased pressure (A) would cause it to disappear. As the curve B may be regarded as fixed, while A, through imperfect maintenance of boiler or engine, may fall, the argument is not strong in favor of increasing pressure beyond the limit of 160 lb.

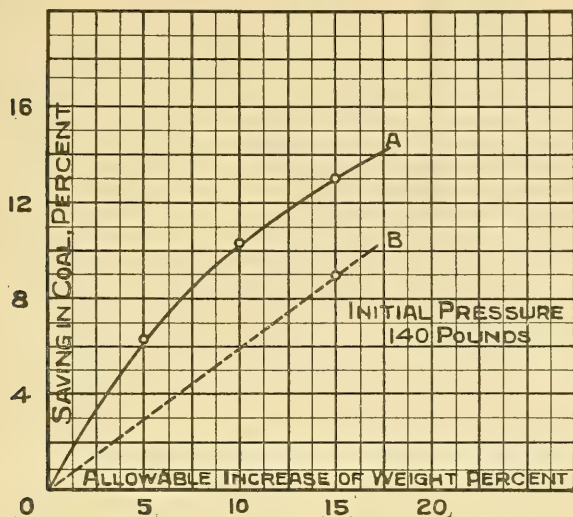


FIG. 17

The line A represents the saving in fuel when an allowable increase in weight is utilized in making a stronger boiler to permit a higher pressure.

The line B represents the saving in fuel when an allowable increase in weight is utilized in making a larger boiler to give increased capacity.

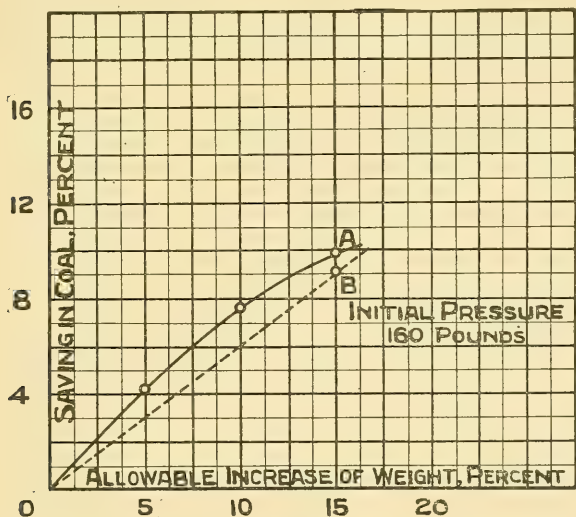


FIG. 18

The line A represents the saving in fuel when an allowable increase in weight is utilized in making a stronger boiler to permit a higher pressure.

The line B represents the saving in fuel when an allowable increase in weight is utilized in making a larger boiler to give increased capacity.

Basing comparisons upon an initial pressure of 180 lb., (Fig. 19), the advantage under ideal conditions of increasing the pressure, as compared with that resulting from increasing the capacity, has a maximum value of approximately one-half of 1 per cent. In view of the incidental losses upon the road the practical value of the advantage is nil. The curves *A* and *B* (Fig. 12),

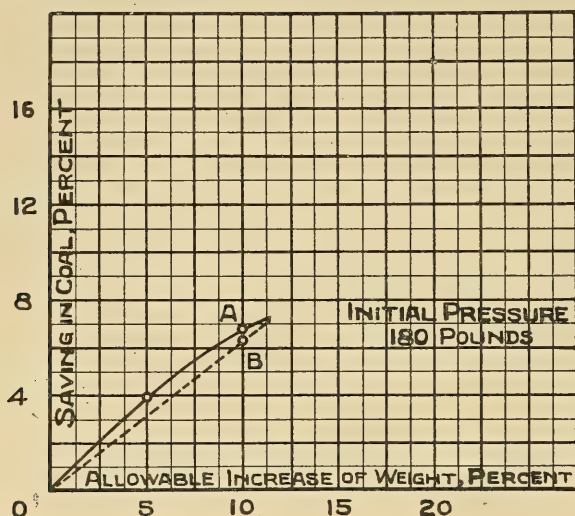


FIG. 19

The line *A* represents the saving in fuel when an allowable increase in weight is utilized in making a stronger boiler to permit a higher pressure.

The line *B* represents the saving in fuel when an allowable increase in weight is utilized in making a larger boiler to give increased capacity.

constitute therefore no argument in favor of increasing pressure beyond the limit of 180 lb.

Basing comparisons upon an initial pressure of 200 lb., (Fig. 20), it appears that under ideal conditions either the pressure or the capacity may be increased with equal advantage, this being in effect a strong argument in favor of increased capacity rather than of higher pressure.

Basing comparisons upon a pressure of 220 lb., (Fig. 21), it appears that even under ideal conditions of maintenance the gain in efficiency resulting from an increase of pressure is less than that resulting from an increase of capacity. In view of this fact, no possible excuse can be found for increasing pressure above the limit of 220 lb.

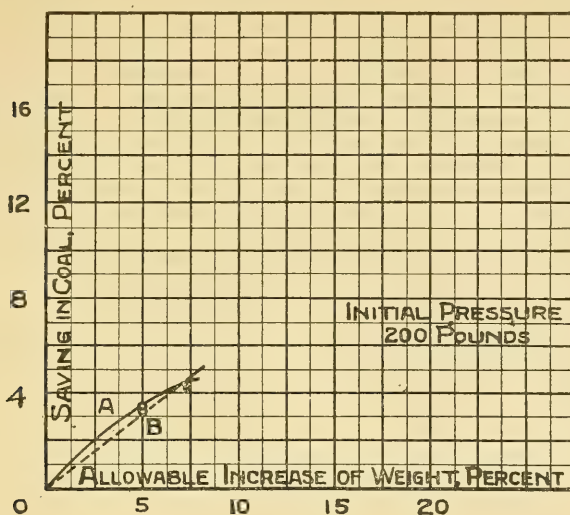


FIG. 20

The line A represents the saving in fuel when an allowable increase in weight is utilized in making a stronger boiler to permit a higher pressure.

The line B represents the saving in fuel when an allowable increase in weight is utilized in making a larger boiler to give increased capacity.

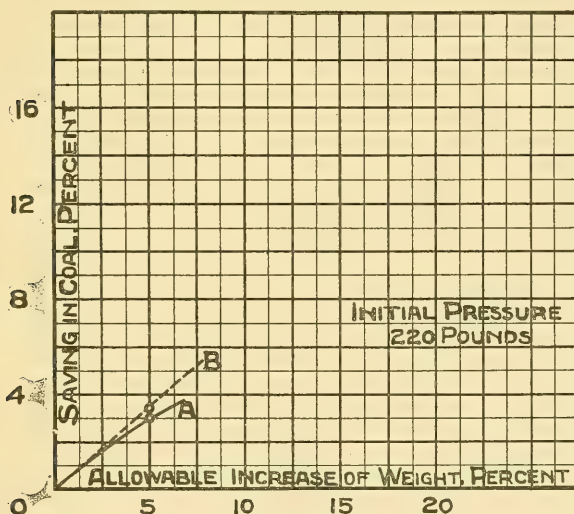


FIG. 21

The line A represents the saving in fuel when an allowable increase in weight is utilized in making a stronger boiler to permit a higher pressure.

The line B represents the saving in fuel when an allowable increase in weight is utilized in making a larger boiler to give increased capacity.

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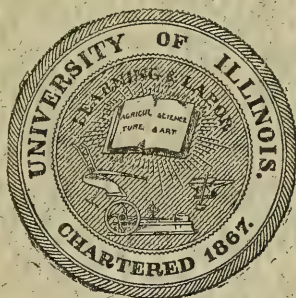
TESTS OF BRICK COLUMNS AND TERRA COTTA BLOCK COLUMNS

BY

ARTHUR N. TALBOT

AND

DUFF A. ABRAMS



UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

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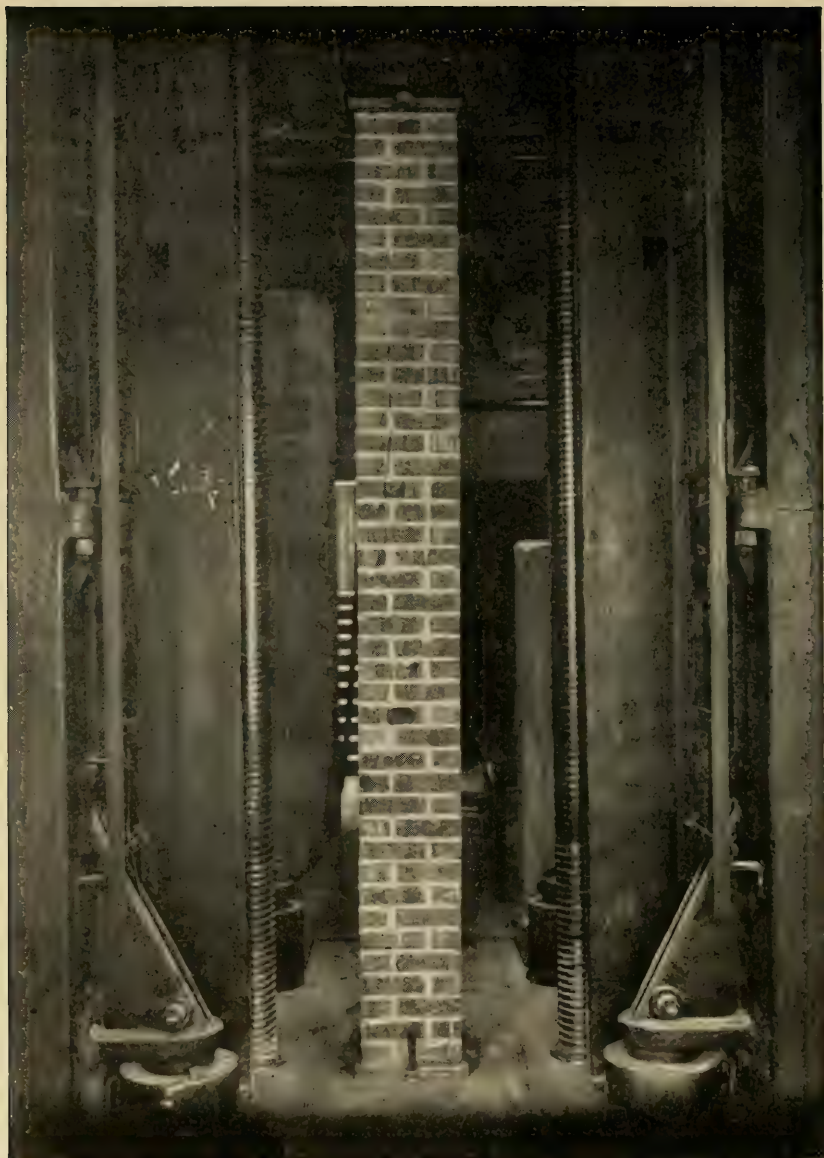
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VIEW OF BRICK COLUMN NO. 61 AFTER TEST, SHOWING
LONGITUDINAL CRACKS.

UNIVERSITY OF ILLINOIS

ENGINEERING EXPERIMENT STATION

BULLETIN No. 27

SEPTEMBER 1908

TESTS OF BRICK COLUMNS AND TERRA COTTA BLOCK COLUMNS

BY ARTHUR N. TALBOT, PROFESSOR OF MUNICIPAL AND SANITARY
ENGINEERING AND IN CHARGE OF THEORETICAL AND APPLIED
MECHANICS, AND DUFF A. ABRAMS, ASSOCIATE IN THEO-
RETICAL AND APPLIED MECHANICS.

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I. INTRODUCTION

1. *Preliminary.*—For brick, stone and terra cotta masonry structures, designers commonly use stresses which are low in comparison with the strength of the constituent materials. Various reasons exist for this,—possible variations in material, chance for poor workmanship, opportunity for settlement and resulting unconsidered stresses, the desire to avoid tensile stresses in masonry, imperfect knowledge of the properties of the completed structure, etc. In some types of structure mass and volume are important considerations and strength is subordinate. In many applications, however, the strength of the structure is of first importance, and a knowledge of the actual properties of the structure is essential to safe and economical construction. Frequently designers use stresses as high as their information and the usual rules of practice will permit and make sections as small as these considerations warrant. The utilization of the working capacity of a structure is generally advantageous. This is especially true when the better grades of material are used and when the workmanship is known to be first-class. Much information is available on the strength and other properties of constituent materials like mortar, brick, stone, and terra cotta, but relatively little is known of the properties of built-up structures. Anything that adds to our knowledge of such construction should be helpful; and it was with a view of obtaining further information on the properties of short built-up columns or piers that the tests herein described were undertaken.

The work described in this bulletin includes tests of 16 short columns or piers of brick and 16 columns built of terra cotta blocks. Although long column action seems not to enter into these compression test pieces, they will be termed columns by analogy to the use of the term column for other compression members of no greater slenderness ratio. The length varied from 10 ft. to $12\frac{1}{2}$ ft. In lateral dimensions the brick columns were $12\frac{1}{2} \times 12\frac{1}{2}$ in., and in the terra cotta block columns the range was from $8\frac{1}{2} \times 8\frac{1}{2}$ in. to $17\frac{1}{2} \times 17\frac{1}{2}$ in. The brick used in the construction of the columns were of two grades,—an excellent class of building brick and a soft grade selected as representative of inferior brick. The terra cotta blocks were of good quality. Different qualities of mortar and different grades of workmanship were used. The loads were

applied continuously to failure in most cases, but tests were also made with repeated applications of the same load. Central loading of the column was used in most cases, but in some tests the load was applied eccentrically.

For convenience of reference in the consideration of the work ~~not~~ described in this bulletin, it has seemed well to mention tests made in other laboratories, and a brief summary of tests of brick piers is given in the following pages. The use of terra cotta blocks in compression members is not common, and there is little in print on the properties of this material.

2. *Watertown Arsenal Tests of Brick Columns.*—The Ordnance Department, United States Army, has from time to time made tests on brick piers at Watertown Arsenal. These tests are the principal source of data on strength of brick columns and the results are valuable. The tests are reported in several of the annual volumes of "Tests of Metals, Etc." Generally, only one test specimen was made for each variation in materials or form of structure. The exact conditions of the tests and the observed data are given in the reports of the tests. The following statement gives a summary of the number of tests and the nature of the investigations.

In "Tests of Metals, Etc." for 1884 are given the results of tests of 33 brick piers ranging from 8 x 8 in. to 16 x 16 in. in lateral dimensions and from 16 in. to 10 ft. in length. Lime mortar, natural cement mortar, and portland cement mortar were used. The results show clearly the increased strength obtained in columns built with the stronger mortar. The compressive strength of the columns ranged from 6 per cent to 16 per cent of the compressive strength of the individual brick. The tests show that columns having joints broken at every course have somewhat less strength than those with the joints broken every sixth course. The tests also show the effect which the load causing the first crack produces on the amount of compression and the amount of set in the column.

In "Tests of Metals, Etc." for 1886 are given the results of tests of 53 brick piers ranging from 8 x 8 in. to 16 x 16 in. in lateral dimensions and from 2 ft. to 12½ ft. in length. Natural cement mortar (1-2) was used. The smaller the ratio of length to lateral dimension the greater was the compressive strength obtained. In cases where bond stones about 3 in. thick were used,

the strength obtained was greater than for a brick column of the same length but was less than for one whose height was equal to the distance between the bond stones. In the columns built with face brick, the strength developed varied from 18 per cent of the compressive strength of the individual brick in a column three diameters long to $12\frac{1}{2}$ per cent in a column fifteen diameters long. In one column test reported in this volume no mortar was used. The column failed in the usual manner by the opening of longitudinal cracks at a load of 525 lb. per sq. in. or about 4 per cent of the strength of the brick. This test indicates the value of the mortar in distributing the load over the surface of the brick.

In "Tests of Metals, Etc.," for 1904, tests of 26 brick piers 12 x 12 in. x 8 ft. are reported. The variation was in the mortar used and in the age. Most of the tests were made at an age of about 6 months. Neat portland cement mortar, 1-2 and 1-3 portland cement mortar, and 1-3 lime mortar were used. The results show the increased strength given by strong mortar. One column laid up with neat cement mortar gave an ultimate strength of 4700 lb. per sq. in.; others with the same mortar but with poorer brick carried less than 2000 lb. per sq. in. The brick used in these tests varied in compressive strength from 4470 to 12 700 lb. per sq. in., based on the average of five samples tested flatwise. The ratios of the strengths of the piers to the compressive strength of the brick were as follows: Neat portland cement mortar, range 23 per cent to 43 per cent, average of 8 tests 30 per cent; 1-3 portland cement mortar, range 16 per cent to 35 per cent, average of 7 tests 24 per cent; 1-3 lime mortar, range 9 per cent to 18 per cent, average of 10 tests 13 per cent. In two tests the 1-3 mortar gave higher values than the neat cement mortar. Some of these test columns were exhibited at the Louisiana Purchase Exposition at St. Louis in 1904.

In "Tests of Metals, Etc." for 1905, tests of 13 clay brick columns and 1 sand-lime brick column are reported. The columns were 12 x 12 in. x 8 ft. high, with solid or hollow cores. The 14 piers were built from 9 different varieties of brick, laid in neat portland cement mortar, cement mortar, and lime mortar. They were tested at an age of about 5 months. The values ranged from 1500 lb. per sq. in. for the sand-lime brick to 4552 for clay brick, laid in neat portland cement mortar. Those laid up with 1-3 mortar gave values from 900 to 3422 lb. per sq. in. No tests of individual brick are given.

In "Tests of Metals, Etc." for 1906 are recorded tests of 15 brick piers 12 x 12 in. x 8 ft. Neat portland cement mortar, 1-3, 1-5, and 1-7 portland cement mortar, and 1-3 lime mortar, and 1 cement to 1 lime mortar were used. Tests were made at about 8 months. The results ranged from 853 to 3437 lb. per sq. in. for clay brick columns and 450 to 1400 lb. per sq. in. for sand-lime brick. In cases where the same brick were used nearly identical values were obtained by using 1-7 portland cement mortar and 1-1 lime mortar.

In "Tests of Metals, Etc." for 1907 are given tests of 32 columns, generally 11 x 11 in. or 12 x 12 in. in lateral dimensions and 8 ft. long. Neat portland cement mortar, 1-1 and 1-3 portland cement mortar, and lime mortar were used. The results show in a very interesting way the effect of the strength of the mortar used. The compressive strength varied from 730 to 5608 lb. per sq. in. The column which developed the highest compressive strength was built with paving brick. The columns built with portland cement mortar gave very high compressive strengths at early ages. One column, built of paving brick laid on edge in neat portland cement mortar, when 4 hours old held 2106 lb. per sq. in.; a similar column 2 days old carried 2733 lb. per sq. in.; and a similar one 7 days old 4514 lb. per sq. in. The columns tested at very early ages showed lack of stiffness and exhibited signs of distress at very low loads. In the column tested at 4 hours' age popping sounds were heard at 400 lb. per sq. in. and a crack appeared at 450 lb. per sq. in.—about 21 per cent of the maximum,—whereas in the columns tested at 2 and 7 days the first cracks appeared at 51 per cent of the maximum load and in a similar column tested at 4 months the first crack came at 61 per cent of the maximum load. Tests in this series indicate that columns in which the bricks are laid on edge or joints broken every third or sixth course give values 8 per cent to 10 per cent higher than columns laid with joints broken every course. Each of these modifications in the method of laying the brick seems to have its effect whether employed separately or in combination.

3. *Cornell University Tests of Brick Columns.*—Two series of tests of brick piers made at Cornell University have been reported. The first series made in 1897-8 consisted of 18 piers 13 x 13 in. in section varying from $2\frac{1}{2}$ to $7\frac{1}{2}$ ft. in length. Two varieties of brick were used. The piers were laid in 1-2 portland cement

mortar, joints broken at each course, and were tested at 4 or 14 months of age. The piers failed under loads of 635 to 1092 lb. per sq. in., corresponding to values 21 per cent to 28 per cent of crushing strength of corresponding brick. Failure generally followed the opening of longitudinal cracks through vertical joints. The variation in length did not affect the load-carrying capacity of the piers. These tests are reported in Transactions of the Association of Civil Engineers of Cornell University, Vol. VI, 1897-8.

In 1898-9 a second series of tests of 14 piers was made. The piers were uniform in size and material,—13 x 13 x 80 in., laid in 1-2 portland cement mortar. The variation was in the form of bonding devices which were placed in the horizontal joints of some of the piers. The brick used gave a crushing strength of 3525 lb. per sq. in. The principal result of these tests was to show the effect of iron or steel straps, netting or plates, embedded in the mortar of the horizontal joints. The maximum loads ranged from 22 per cent to 46 per cent of the crushing strength of the brick. Following is a summary of the results for different conditions in per cent of the crushing strength of the brick: 1-2 portland cement mortar only, (3 tests) 30 per cent; iron straps ($1\frac{1}{4}$ x $\frac{1}{8}$ in.) every fourth course, (2 tests) 24 per cent; iron straps every sixth course, (1 test) 22 per cent; iron straps every eighth course, (1 test) 24 per cent; wire netting ($\frac{1}{2}$ in. square mesh, 0.045 in. wire) every second course, (2 tests) 33 per cent; wire netting every course, (2 tests) 46 per cent; iron plate (0.026 in. thick) every fourth course, (4 tests) 28 per cent. These tests are reported in Transactions of the Association of Civil Engineers of Cornell University, Vol. VIII, 1899-1900.

4. *School of Practical Science Tests of Brick Columns.*—During the years of 1895 and 1896, 17 short brick piers were tested at the School of Practical Science, Toronto. These piers were 9 x 9 in. in section and ranged from 21 to 73 in. in height. Eleven grades of brick were used. 1-2 lime mortar and 1-3 cement mortar were used. The age at time of test was generally $2\frac{1}{2}$ months; one test was made at ten days. Careful records of strength of mortar and properties of brick were kept. Of the piers loaded to failure those built of lime mortar (11 tests) gave values averaging 16 per cent of the strength of the individual brick loaded flatwise, while those built of cement mortar (5 tests) gave values averaging 46 per cent of the strength of the brick. For further details

of these tests see Digest of Physical Tests, Vol. I, No. 3, July, 1896.

5. *Acknowledgment.*—The investigation was made at the instance of architects and builders who were desirous of finding the properties of columns built with material manufactured in this state. Mr. F. W. Butterworth of the Western Brick Company, of Danville, Illinois, furnished the building brick. The terra cotta blocks were supplied by the National Fire Proofing Co., of Chicago. The tests were made in the Laboratory of Applied Mechanics of the University of Illinois as a part of the regular work of the Engineering Experiment Station.

II. MATERIALS, TEST PIECES, AND METHODS OF TESTING.

6. *Brick.*—Two varieties of brick were used in these tests. The columns numbered 51 to 72 inclusive were built of high-grade, hard-burned, shale building brick, manufactured by the Western Brick Co., of Danville, Illinois. Columns No. 81 and 82 were built of soft, under-burned clay brick manufactured by the Sheldon Brick Co., of Champaign, Illinois, and selected from the soft brick of the kiln. The two varieties of brick represent extremes in quality. The soft brick was softer than would be used in building construction. This extreme was chosen better to bring out the effect of quality. The two varieties of brick will be referred to hereafter as “shale building brick” and “underburned clay brick” or “shale brick” and “clay brick”. The results of compression tests of individual brick are given in Table 1, page 10. In all compression tests the specimen was first bedded in plaster of paris. In order to obtain an independent check on the quality of the brick used, transverse tests were made from which the modulus of rupture was computed. The results of these tests are given in Table 2, page 11. The tests of the brick in compression and cross-breaking gave results as uniform as are ordinarily found for this kind of material. The ratio of the modulus of rupture of brick tested edgewise, to the compressive strength of whole brick tested flatwise is 0.155 for the shale brick and 0.123 for the clay brick.

Fig. 1, page 9, gives the rate of absorption and the final amount of water absorbed by the two varieties of brick and by the terra cotta blocks. Each curve represents the average of 10

samples selected at random from the material furnished. The bricks and blocks were dried for about 48 hours on a steam radiator before the initial weight was taken. The same brick and blocks were afterward used in compression or transverse tests.

7. *Terra Cotta Blocks*.—The terra cotta blocks were furnished by the National Fire Proofing Co., of Chicago. The blocks came in two shipments. Columns No. 1 to 7, (tested January, 1907) were made from the first shipment, and the remainder of the terra cotta block columns from the second shipment. There was apparently no difference in the blocks in the two shipments, but both inspection and tests showed that the blocks of the second lot were more perfectly cut. In the first lot the ends or bearing faces were concave; in the second lot they were more nearly plane.

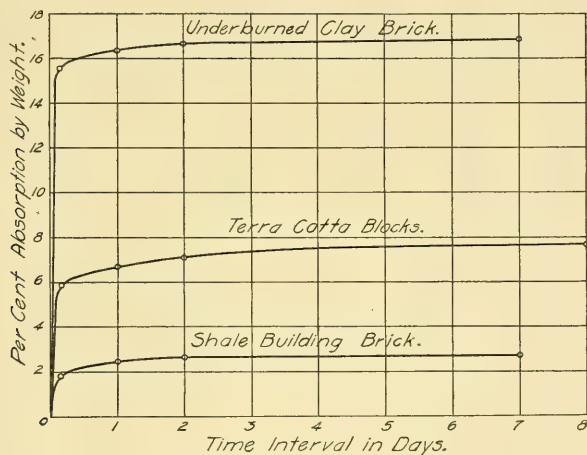


FIG. 1. ABSORPTION TESTS OF COLUMN MATERIALS.

Attention will be called later to the significance of this statement. It seems probable that the blocks represent a selected quality of the output of the factory. The size of the blocks was about 4 in. x 8½ in. x 8 in. A small quantity in the second shipment were 4 in. x 8½ in. x 4 in. The results of the tests of the small blocks in compression and in cross-breaking are included with the others, although none of them were used in building the columns. A sketch of a block with dimensions is given in Fig. 2, page 14. The gross section of each block was about 34 sq. in. and the net section about 28.75 sq. in., or 84.6 per cent of the gross section. The results of compression tests of individual blocks are given in Table 3, page 12.

TABLE 1.
COMPRESSION TESTS OF BRICK.

Ref. No.	Dimensions, inches			Dry Weight pounds	Ultimate Load pounds	Compressive Strength lb. per sq. in.
	Width	Length	Depth			
Shale Building Brick						
1	3.95	8.25	2.35	6.36	364 000	11 150
2	3.90	8.20	2.42	6.48	312 000	9 750
3	3.92	8.25	2.40	6.42	259 000	8 020
4	3.85	8.25	2.35	6.46	223 000	7 030
5	3.95	8.15	2.32	6.00	320 000	9 950
6	4.00	8.32	2.35	6.28	365 000	11 000
7	3.85	8.25	2.30	6.01	320 000	10 090
8	3.85	8.20	2.30	6.10	349 000	11 050
9	3.95	8.20	2.33	6.11	264 000	8 150
10	3.95	8.25	2.30	6.56	462 000	14 150
11	4.00	8.34	2.30	6.29	457 000	13 700
12	3.92	8.30	2.37	6.50	348 000	10 700
13	3.90	8.25	2.40	6.51	359 000	11 150
14	3.90	8.20	2.30	6.51	453 000	14 150
15	3.95	8.30	2.40	6.44	378 000	11 520
16	4.05	8.35	2.25	6.36	373 000	11 020
17	4.00	8.30	2.40	6.40	324 000	9 750
18	3.90	8.28	2.43	6.49	324 000	10 040
Av.						10 690
Underburned Clay Brick						
1	4.00	8.25	2.40		134 300	4 070
2	4.05	8.35	2.40		92 800	2 750
3	4.15	8.30	2.43		99 200	2 870
4	4.00	8.00	2.45		110 400	3 450
5	3.75	7.60	2.45		238 900	8 390
6	4.05	8.25	2.40		73 200	2 190
7	4.05	8.30	2.30		108 300	3 230
8	4.00	8.00	2.40		107 900	3 370
9	4.00	8.20	2.45		159 000	4 850
10	4.07	8.40	2.55		83 200	2 430
12	4.10	8.10	2.45		175 700	5 290
13	4.15	8.25	2.40		95 500	2 790
14	4.10	8.20	2.35		119 600	3 560
15	4.00	8.00	2.30		181 500	5 670
16	4.00	8.00	2.40		125 900	3 930
Av.						3 920

All brick were bedded in plaster of paris before testing.

Seven shale brick tested on end averaged 7800 lb. per sq. in.

Seven clay brick tested on end averaged 3530 lb. per sq. in.

TABLE 2.

TRANSVERSE TESTS OF BRICK.

Ref. No.	Dimensions inches		Total Center Load pounds	Modulus of Rupture lb. per sq. in.	Remarks on Failure
	Depth	Width			
Shale Building Brick					
1	3.96	2.38	4900	1180	Near middle.
2	3.86	2.39	6100	1540	do.
3	3.97	2.38	4100	963	do.
4	3.87	2.45	5600	1385	do.
5	3.85	2.37	5700	1450	At middle.
6	3.94	2.44	7600	1800	do.
7	3.95	2.48	8000	1860	do.
8	3.90	2.40	8000	1970	Near middle.
9	3.80	2.35	7300	1940	Diagonal break.
10	3.92	2.40	7300	1780	Angular break near middle.
Av.				1670	
Underburned Clay Brick					
1	4.00	2.40	2000	516	Near middle.
2	4.08	2.48	1900	413	do.
3	4.00	2.48	2250	510	do.
4	4.05	2.45	1300	292	do.
5	3.95	2.40	2400	578	Near middle. Very regular.
6	4.00	2.45	2800	643	At middle.
7	4.05	2.40	2100	480	Near middle.
8	4.00	2.40	2000	469	do.
9	4.02	2.40	2100	489	At middle.
10	4.06	2.46	1900	423	do.
Av.				481	

Notes: The bricks were tested on edge, with a span of 6 in. The load was applied at the middle of the span, with a machine speed of 0.02 in. per minute. Curved pedestals, standing on a wooden block, were used. Metal bearing plates distributed the load across the surface of the brick at all points.

TABLE 3.
COMPRESSION TESTS OF TERRA COTTA BLOCKS.
Blocks loaded on end.

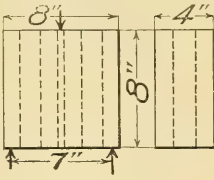
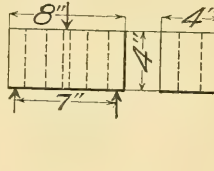
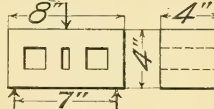
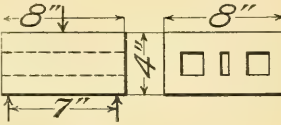
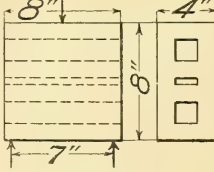
Ref. No.	Dimensions, inches		Dry Weight pounds	Total Load pounds	Compressive Strength lb. per sq. in.		Remarks
	Section	Height			on gross area	on net area	
Blocks as used in columns tested January, 1907							
1	4.10×8.50	8.00	17.12	99 000	2840	3350	Each end concave 1-16 in.
2	4.10×8.45	8.00	17.16	135 000	3900	4600	Each end concave $\frac{1}{8}$ in.
3	4.05×8.45	8.00	17.03	124 000	3630	4290	Ends concave $\frac{1}{8}$ to 1-16 in.
4	4.00×8.45	8.00		155 000	4580	5410	Ends fairly straight.
5	4.10×8.55	8.00	17.05	108 000	3080	3640	Ends concave 1-16 to $\frac{1}{8}$ in.
6	4.10×8.52	8.00	17.20	108 000	3090	3650	Each end concave $\frac{1}{8}$ in.
7	4.05×8.45	8.00	16.98	110 000	3220	3800	Ends concave 1-16 to $\frac{1}{8}$ in.
8	4.10×8.45	8.00	16.62	119 000	3440	4060	Ends straight.
Av.					3472	4100	
Blocks as used in columns tested January, 1908.							
1	3.95×8.25	8.00	15.82	194 000	5980	7100	One end concave 3-32 in.
2	3.92×8.23	7.92	15.65	232 000	7480	8590	Ends straight.
3	3.94×8.23	8.00	15.88	214 000	6550	7770	1-16 in. on one end, other straight.
4	3.96×8.30	8.00	15.92	251 000	7635	9070	One straight, one concave 1-16 in.
5	3.96×8.25	7.90	15.63	145 000	4440	5250	One end concave 1-16, one convex 1-16 in.
6	3.95×8.20	8.00	15.74	182 700	5640	6660	do.
7	4.00×8.25	8.00		127 500	3860	4560	do.
8	3.90×8.20	7.95		174 200	5450	6450	do.
9	4.00×8.35	8.00		116 700	3490	4120	do.
10	3.90×8.30	8.00	16.07	156 000	4820	5700	One end straight, one end concave 1-16 in.
11	4.00×8.25	8.00	15.64	122 000	3700	4380	do. $\frac{1}{8}$ in.
12	4.00×8.30	8.00	15.92	102 000	3070	3630	do.
13	3.90×8.25	8.00	15.69	108 000	3350	3960	do.
14	3.90×8.20	8.00	15.88	205 000	6420	7560	do.
15	4.00×8.35	8.00	15.63	180 000	5390	6360	do. 3-32 in.
16	4.00×8.30	8.00	14.70	123 800	3720	4400	One concave $\frac{1}{8}$ in. twisted, one convex 1-16 in.
17	3.90×8.20	8.10	15.93	218 700	6850	8100	One straight, one concave $\frac{1}{8}$ in.
18	3.95×8.30	8.00	15.62	177 000	5400	6380	do.
Av.					5170	6115	
1	4.05×8.50	3.95	8.28	168 700	4900	5800	
2	4.05×8.50	4.05	8.30	162 400	4720	5580	
3	4.00×8.45	4.05	8.20	125 000	3700	4370	
4	4.05×8.50	4.05	8.28	136 300	3960	4680	
5	4.05×8.45	4.00	8.39	158 000	4620	5460	
6	4.00×8.40	4.00	8.21	169 400	5040	5950	
7	4.05×8.40	4.10	8.28	149 800	4400	5200	
Av.					4475	5290	

Notes: All blocks were bedded in plaster of paris before loading. The dimensions given are over all, and include two openings, $1\frac{1}{2}$ in. x $1\frac{1}{2}$ in., and one opening $\frac{1}{2}$ in. x $1\frac{1}{2}$ in. (See sketch of block, Fig. 2 page 14.) Tests were made on a 600 000-lb. testing machine; speed 0.05 in. per minute.

TABLE 4.

TRANSVERSE TESTS OF TERRA COTTA BLOCKS.

Blocks as used in columns tested January, 1908.

R ef. N o.	Dimensions inches		Span in.	Total Center Load lb.	Modu- lus of Rup- ture lb. per sq. in.	Remarks	Method of Loading
	Width	Depth					
1	4.0	8.0	8	12 700	970	Medium color	
2	3.95	8.0	8	11 300	870	do.	
3	4.1	8.0	8	14 000	1010	Very light in- side	
4	4.0	8.0	8	11 700	880	Light inside	
5	4.0	8.0	8	16 300	1220	Medium color	
6	4.0	8.0	7	18 900	1240	Dark inside	
7	4.0	8.0	7	18 000	1180	do.	
Av.					1022		
1	4.0	4.0	8	2 500	750		
2	4.0	4.0	8	3 600	1080		
3	4.0	4.0	8	4 800	1440		
4	4.0	4.0	7	5 100	1340	Broke $\frac{1}{2}$ in. from load	
5	4.0	4.0	7	3 200	841	Broke near mid- dle	
6	4.0	4.0	7	3 700	974	do.	
7	4.0	4.0	7	4 900	1290	do.	
Av.					1102		
1	4.0	4.0	7	4 000	690	Broke through outer opening	
2	4.0	4.0	7	3 300	578	do.	
3	4.0	4.0	7	3 600	620	do.	
Av.					629		
1	8.25	4.0	7	11 200	912	Broke at mid- dle. Light in- side	
3	8.4	4.0	7	9 700	770	Break very reg- ular	
3	8.35	4.0	7	8 800	708	do.	
Av.					797		
1	4.0	8.25	7	13 000	562	Diagonal fail- ure	
2	4.0	8.25	7	16 300	707	do.	
Av.					635		

NOTES: In calculating the modulus of rupture the net section of the block was used, a deduction for the opening being made. The load was applied at the middle of the span in each case. Curved pedestals, standing on a wood block, were used. Metal bearing plates distributed the load across the block at all points. The pulling head of the machine moved at a rate 0.02 in. per minute in all these tests.

The crushing load on the block is given as computed for both gross and net areas. It will be noted that the condition of the ends recorded in the table has a decided influence on the strength of the block in this kind of a test. In the compression tests the terra cotta blocks began to spall at the corners at about 90 per cent of the ultimate load. As the ultimate load was approached, popping noises were heard and failure soon followed by the block being completely shattered by the formation of numerous longitudinal cracks. All the tests given were made at the same time under identical conditions. Results of transverse tests on terra cotta blocks are given in Table 4, page 13. These tests were made with blocks loaded on top, on side and on edge. The method of loading is indicated by sketches in the table.

Fig. 1, page 9, gives the rate of absorption as already described.

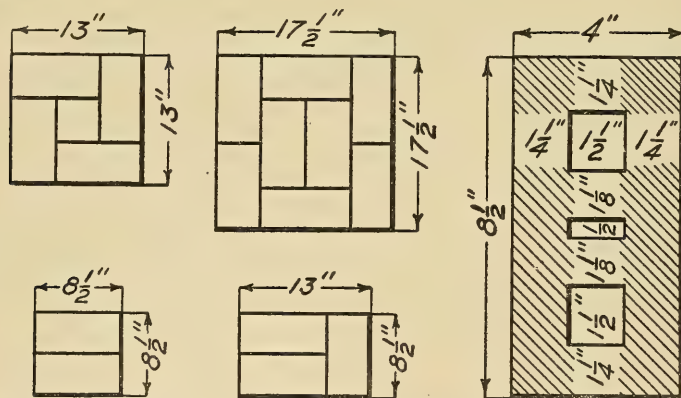


FIG. 2. SKETCH OF A TERRA COTTA BLOCK AND ARRANGEMENT OF TERRA COTTA BLOCKS AND BRICK IN VARIOUS SECTIONS.

8. *Cement, Sand, and Mortar.*—Universal portland cement mortar was used in all 16 terra cotta block columns and in 13 of the brick columns. In two brick columns lime mortar was used and in one brick column Bricklayers' (natural) cement was used. The results of tensile tests of these cements are given in Table 5, page 15. The results of the test of the Universal cement for the two different years are given separately. The cement for the terra cotta block columns tested January, 1907 was from the 1907 lot; all other portland cement used was from the 1908 lot. The

sand was the same as was used for the reinforced concrete tests during the season of 1906-7. It came from near the Wabash river at Attica, Indiana. It was screened through a No. 10 sieve and contained a rather large amount of clay. Average values from mechanical analyses of the sand are given in Table 6, page 16.

TABLE 5.
TENSILE STRENGTH OF CEMENT.

Ref. No.	Ultimate Strength, lb. per sq. in.											
	1907 Universal (portland)				1908 Universal (portland)				Bricklayers' (natural)			
	Age 7 days		Age 28 days		Age 7 days		Age 28 days		Age 7 days		Age 28 days	
	Neat	1-3	Neat	1-3	Neat	1-3	Neat	1-3	Neat	1-3	Neat	1-3
1	410	187	680	370	563	244	764	319	120	40	175	87
2	470	200	670	330	809	248	885	336	120	38	210	87
3	360	120	560	360	728	232	776	285	100	40	130	81
4	405	145	570	290	699	242	754	292	115	35	140	84
5	320	195	600	295	702	229	763	315	100		190	87
Av.	379	171	617	326	700	239	788	309	111	38	169	85

The 1-3 briquettes were made from standard Ottawa sand.

All the values for each cement given in any horizontal line are from briquettes made from the same sample. Each value given for the Universal cement is the average of 5 briquettes.

The values given for the Bricklayers' cement are each from a single briquette test. All the tests are from a single sample of cement.

A 1-2 mortar was used in making 7 terra cotta block columns, a 1-3 mortar in 7 columns and a 1-5 mortar in 2 others. Of the brick columns, 11 were made with 1-3, and 2 with 1-5 portland cement mortar, 1 with 1-3 Bricklayers' (natural) cement mortar, and 2 with 1-2 lime mortar. The proportioning was by loose volume in all cases. The mortar was mixed in small batches as needed. The mixing was done by a bricklayer's helper, who used the ordinary methods for such work. In most cases as the columns were laid up, one or two 6-in. cubes were made from each batch of cement mortar and afterwards tested as a check on the quality of the mortar. The results of these tests may be found in Table 9, page 26, and Table 13, page 42. The value given for each column is the average of the tests of the cubes made from the mortar used in the corresponding column.

9. *Dimensions of Columns.*—The first 7 terra cotta block columns (those tested January, 1907) were built from 11 ft. 10 in. to 12 ft. 6 in. high and of four different cross sections, $8\frac{1}{2}$ in. x $8\frac{1}{2}$ in., $8\frac{1}{2}$ in. x 13 in., 13 in. x 13 in., and $17\frac{1}{2}$ in. x $17\frac{1}{2}$ in. The remainder of the columns, both terra cotta block and brick, were built about 10 ft. high and with a cross section about $12\frac{1}{2}$ in. x $12\frac{1}{2}$ in. The exact dimensions of the columns are given in Tables 7 and 11, pages 24 and 39. The arrangement of the blocks in the different sections is indicated in Fig. 2, page 14.

TABLE 6.

MECHANICAL ANALYSIS OF SAND.

Average of 5 Samples.

Sieve No.	Separation Size inches	Per cent Passing
10	.091	100
12	.067	87.3
16	—	80.0
18	.043	65.0
30	.027	42.7
40	.019	28.5
50	.013	17.3
74	.009	8.3
150	—	2.3
200	—	1.6

10. *Fabrication and Storage of the Columns.*—The columns were made in the Laboratory of Applied Mechanics of the University of Illinois. The work was done by experienced bricklayers. Columns No. 1 to No. 7, inclusive, were laid up by a workman sent for that purpose by the National Fire Proofing Co., of Chicago, the company which furnished the terra cotta blocks for these tests. This workman had had considerable experience in this particular kind of work and had previously built a number of columns for testing purposes. The remainder of the terra cotta block and all of the brick columns were built by a workman furnished by a local contractor, who was requested to select a representative bricklayer. He had had no previous experience in laying terra cotta blocks, but was thrown upon his own resources after receiving necessary instructions regarding dimensions, etc. The work of the latter man seems to be more nearly representative of what would be obtained in an ordinary case, although no

appreciable difference in the results can be traced to the difference in workmanship. No special care was taken in selecting the brick or blocks in building the columns except that from the second shipment of terra cotta blocks enough under-burned or apparently inferior blocks were set aside to build one column, (No. 17).

A base plate of cast iron about $1\frac{1}{4}$ in. in thickness, planed on each side, was leveled on the concrete floor and the column built upon it. The terra cotta blocks and clay brick were plunged into a bucket of water a few minutes before laying. The shale brick were so impervious that this was not necessary. All columns described herein were laid up solid, i.e., no hollow spaces were left, except the vertical openings in the terra cotta blocks. No attempt was made to fill these openings with mortar. Joints were broken in every course in all columns built. The brick were laid flatwise as in practice. The number of courses and the thickness of the mortar joints are given in the tables for each column. A cast-iron bearing plate similar to the one mentioned above was bedded in mortar on the top of each column. These plates were useful when moving the columns to the testing machine, and they served as bearing plates in the test. The columns composing a set (similarly built and tested) were built on different days in order that average conditions might govern and thus eliminate as far as may be the accidental effect of the conditions of workmanship and materials.

All of the columns which are designated in the tables as well laid were built with the care usually given to first-class masonry. In building five of the columns (brick columns No. 55 and 56, and terra cotta block columns No. 12, 13, and 14) designated in the tables as poorly laid, the workman was told to use less care in making horizontal joints and in filling the vertical joints. It was the intention to produce the effect of a hurried or careless workman. These columns were built in about three-fourths the time required for a similar one well laid.

Each column was left standing in the place in which it was built until time of test. The columns were kept in an upright position as they were transported to the testing machine by means of an overhead crane. No attempt was made to prevent the mortar in the joints from drying out. The temperature of the room in which the columns were made and stored varied from 55° to 70° F.

The columns were tested at an age of about 60 days. Two brick columns were tested at an age of 6 months. The exact age at time of test is given for each column.

11. *Methods of Testing.*—Fourteen terra cotta block columns were tested with concentric loading and two with eccentric loading; on some of the columns the load was applied in more than one way. One of the terra cotta block columns (No. 15) did not fail after several repetitions of the concentric load up to the capacity of the machine and was afterward loaded eccentrically until failure occurred. Terra cotta block columns No. 7 and No. 8 did not fail after receiving several applications of the full load of the testing machine. No. 7 was loaded eccentrically also but not to failure. They were set aside for further test at some future date. Two of the terra cotta block columns, No. 6 and No. 7, had a gradually increasing load applied and released in succession. Of the brick columns, 14 were tested with concentric load and 2 with the load applied eccentrically. In the test of No. 60 (shale brick, 1-3 mortar, age at test 6 months) failure was finally produced by repeating the maximum load a number of times.

In several cases when failure occurred at or near the top of the column enough of the top was removed so that the remaining portion contained no cracks, a bearing plate was placed in mortar on the top, and the resulting short column afterward tested. The results of these tests do not differ greatly from those of the original columns. The slight increase in compressive strength may be accredited to the increase in the strength of the mortar during the interval.

All the tests were made in the 600 000-lb. Riehle vertical screw testing machine in the Laboratory of Applied Mechanics of the University of Illinois. The machine head moved at the rate of 0.05 inches per minute; except in some cases, where the load was repeated, when a speed of 0.10 inches per minute was used. The load was generally applied by increments of about 25 000 lb. on the column, but for the columns built of lean mortar or of clay brick the increment was smaller. The load was applied through a spherical bearing block, except where the column was loaded eccentrically. In arranging the columns for eccentric loading, a $\frac{1}{2}$ -inch square steel bar about twenty inches long was placed one inch off the center of the column under the lower bearing block, and a similar bar in a corresponding position on top of the upper

bearing block. The load was applied to the column through these bars.

The measurements of longitudinal deformations were made by instruments which are shown in the views in Fig. 8. The yokes were so placed on the column that measurements were taken over a length of about 100 inches. On each side of the lower yoke an instrument was placed which read direct on the dial to thousandths and by estimation to ten-thousandths of an inch. After each increment of load was applied, the machine was stopped and readings of the extensometers taken before again increasing the load. Readings were generally taken at increments of load of 25 000 lb., although smaller increments were used in the tests of the columns in which clay brick or a lean mortar was used.

In some of the earlier tests the lateral deflection of the column from its original vertical position was measured by means of two fine silk threads and mirror scales placed on two faces near the middle of the column. The threads were fastened at points near the ends of the column and held taut. The amount of the center deflection would be indicated by the movement of the thread over the scale. The actual amount and direction of the deflection could be calculated by considering the scale readings as the rectangular components of the movement. The measurement of lateral deflection was finally discontinued, as the total amount of the movement was so small as to be insignificant. The total center deflection of a brick or terra cotta block column $12\frac{1}{2} \times 12\frac{1}{2}$ in. in section, and 10 ft. long, in which the stronger mortar was used, was no greater than 0.03 in. to 0.08 in.

III. EXPERIMENTAL DATA AND DISCUSSION.

A. BRICK COLUMN TESTS.

12. *Brick Column Test Data.*—Fourteen columns of shale building brick and two columns of underburned clay brick, all approximately $12\frac{1}{2} \times 12\frac{1}{2}$ in. in cross section and 10 ft. long, were tested. The make-up of the columns, method of applying the load and the age at test varied. Table 7, page 24, gives data on the dimensions and make-up of the brick columns. Table 8, page 25, gives data on the tests of the columns.

13. *Phenomena of the Tests of Brick Columns.*—As already stated, the brick columns were made in sets of two or three,

which were constructed and loaded similarly. With the exception of No. 60, these columns were loaded continuously to failure. In the test of No. 60, a load above the rated capacity of the machine was released and re-applied seven times before failure finally occurred. The phenomena of the continuous-loading tests will be described here, and the repeated-load test will be discussed in a succeeding paragraph.

In all of the brick column tests, the first evidence of distress observed was a faint popping sound which seemed to proceed from the interior of the column. The load at which this popping was first heard, expressed as a proportional part of the maximum load carried by the column, is given in the last column of Table 8, page 25. As the load was increased, the popping noises were heard more frequently and were louder. As the load was further increased, the action of the columns depended to a great extent on the quality of the mortar used. The columns in which the richer and stronger mortar was used gave little or no additional evidence of distress until a load a little below the maximum was reached, when spalling of the mortar at the corners of the column, or the formation of longitudinal cracks through the vertical joints gave warning of impending failure; after the beginning of spalling or the formation of longitudinal cracks was observed, the failure was generally very sudden and complete. In this class of columns (those with 1-3 portland cement mortar) the debris showed that failure was precipitated by the formation of longitudinal cracks through each vertical joint. These cracks generally extended throughout about the upper two-thirds of the length of the column, beginning near the middle and extending both ways. Such failures were extremely violent and sometimes involved hazard to observers and often proved destructive to measuring instruments. The failures came with such slight warning that, despite repeated efforts, the operator was unable, except in two tests (Columns No. 55 and 56), to stop the movement of the testing machine after evidences of failure were observed in time to avoid reducing the upper two-thirds of the column to a mass of broken bricks and mortar scattered over the Laboratory. The design of this machine is such that it may be stopped and the motion completely reversed almost instantly. It will be found by reference to the table that the columns referred to were the ones laid up carelessly. They were less rigid than the others of the same materials and hence took on load more

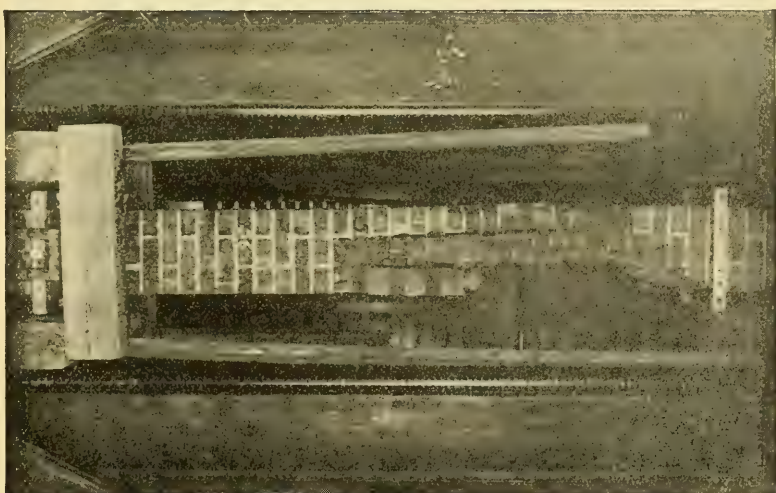
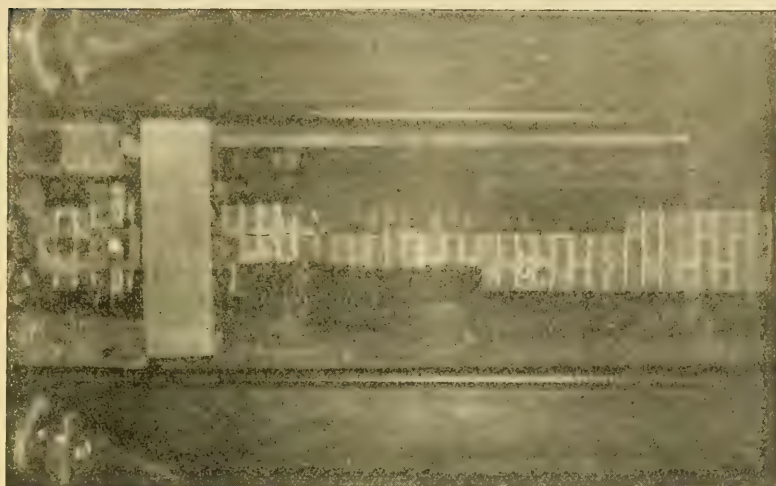


FIG. 3 VIEWS OF BRICK COLUMNS NO. 55 AND 56 AFTER TESTS SHOWING MANNER OF FAILURE.

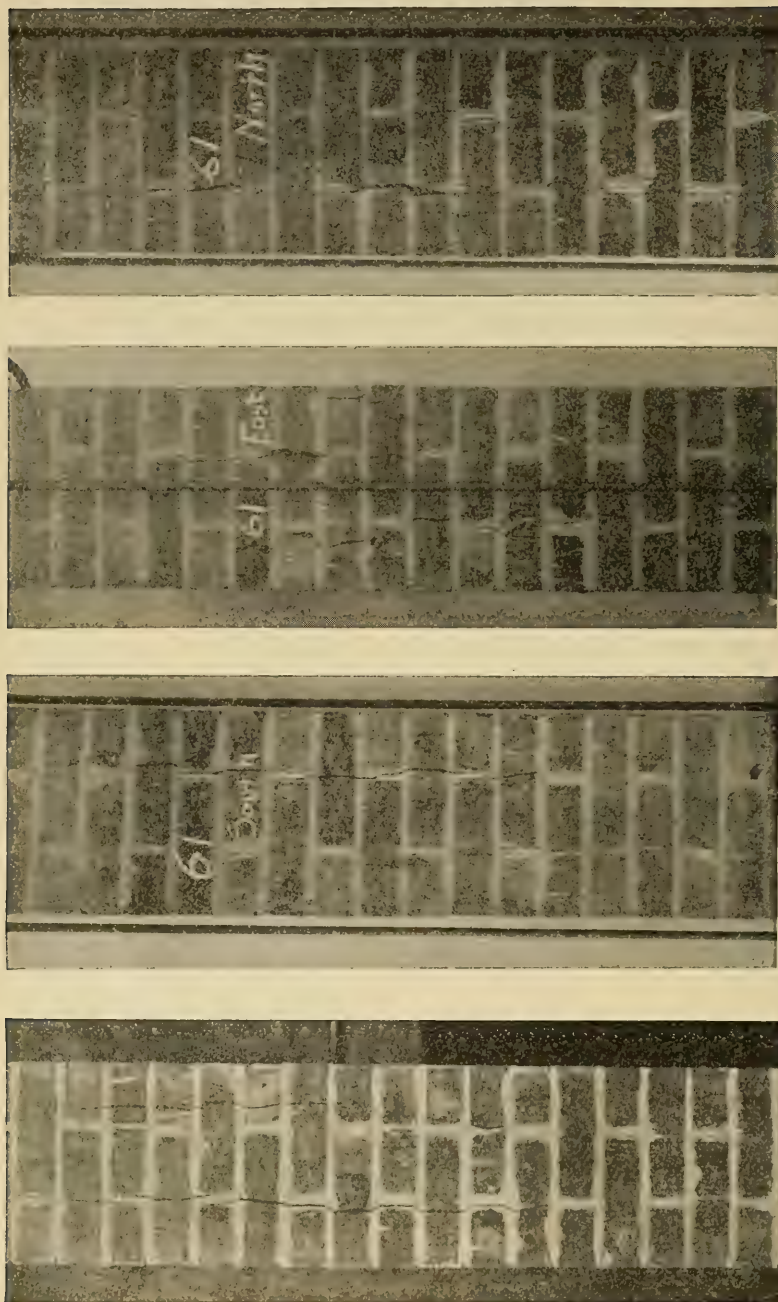


FIG. 4. VIEWS OF BRICK COLUMN No. 61 AFTER TEST SHOWING CRACKS FORMED ON EACH OF THE FOUR FACES.

slowly. The photographs, Fig. 3, page 21, show the two brick columns (No. 55 and No. 56) in which the collapse was not complete; the smallest section of No. 55, about $2\frac{1}{2}$ ft. below the top, is scarcely larger than one brick.

The phenomena of the test of the clay-brick columns did not differ greatly from those of shale-brick with 1-3 portland cement mortar, except that on account of the reduced rigidity the load was applied more slowly and the failures were less sudden and violent.

The failures of the columns in which 1-5 portland cement, natural cement, or lime mortar was used were similar to those described above, except that they rarely got beyond the spalling and cracking stage. In these tests the formation of the vertical cracks could readily be observed. The freedom from sudden collapse was probably due to the yielding of the joints and the fact that the testing machine does not follow such yielding instantaneously. It must not be inferred that under an actual load such columns would not have failed as suddenly and completely as the others. Where natural cement or lime mortar was used, the mortar gradually disintegrated and reduced to powder.

The photographs reproduced in Fig. 4, page 22, show the size and location of these vertical cracks in a brick column in which 1-5 portland cement mortar was used. Each view shows the condition of the upper half of one face of the column after failure. Some of the cracks could be traced nearly to the bottom of the column. This distribution of cracks is characteristic of the columns with stronger mortar, but, as stated above, it was impossible to discontinue the tests of the stronger columns at this stage. In these tests also the cracks and excessive spalling were generally confined to the upper two-thirds of the length of the column.

This phenomenon may be accounted for by the consideration that the mortar in the lower portion of the column is appreciably stronger than that above, due to the weight of the column coming upon it during the period of setting. This phenomenon has been observed in other tests designed for that purpose, both in this Laboratory and elsewhere. Recently two sets of 6-in. cubes (3 cubes in a set) were made from the same batch of concrete. Two cubes from each set were allowed to harden in the open air as usual, while the third was loaded with a pressure of about 10 lb.

per sq. in. The cubes were tested in compression after a period of 10 days; the cubes which set under pressure giving in each case a value of about 33 per cent in excess of the average of the other two.

The columns loaded eccentrically failed by the formation of vertical cracks parallel to the loading plane. They were thrown from the machine toward the side opposite the load.

TABLE 7.
DATA OF BRICK COLUMNS.

Col. No.	Characteristics of Column and Loading	Kind of Mortar	Length ft.-in.	Nominal Section in. x in.	Gross Area sq. in.	Number of Courses	Average Thickness of Joint inches
Shale Building Brick							
51	Well laid, concentric load.	1-3 port. cement	9-10	$12\frac{1}{2} \times 12\frac{1}{2}$	157.5	43	.33
52	do.	do.	9-11 $\frac{1}{2}$	$12\frac{1}{2} \times 12\frac{1}{2}$	158.8	43	.37
53	do.	do.	9-9	$12\frac{1}{2} \times 12\frac{1}{2}$	156.3	42	.38
59	do.	do.	9-11	$12\frac{1}{2} \times 12\frac{1}{2}$	158.8	43	.36
60	do.	do.	9-11	$12\frac{1}{2} \times 12\frac{1}{2}$	158.1	43	.36
55	Poorly laid, concentric load.	do.	9-11	$12\frac{1}{2} \times 12\frac{1}{2}$	158.1	43	.36
56	do.	do.	10-0	$12\frac{1}{2} \times 12\frac{1}{2}$	155.0	43	.38
57	Well laid, eccentric load. ($\epsilon = 1$ in.)	do.	9-10	$12\frac{1}{2} \times 12\frac{1}{2}$	156.9	43	.33
58	do.	do.	9-11 $\frac{3}{4}$	$12\frac{1}{2} \times 12\frac{1}{2}$	156.9	43	.38
61	Well laid, concentric load.	1-5 port. cement	9-9 $\frac{3}{4}$	$12\frac{1}{2} \times 12\frac{1}{2}$	159.4	42	.39
62	do.	do.	9-10	$12\frac{1}{2} \times 12\frac{1}{2}$	158.1	42	.40
91	do.	1-3 natural cement	10-0	$12\frac{1}{2} \times 12\frac{1}{2}$	158.8	43	.38
71	do.	1-2 lime	9-10	$12\frac{1}{2} \times 12\frac{1}{2}$	158.8	43	.33
72	do.	do.	9-8 $\frac{1}{2}$	$12\frac{1}{2} \times 12\frac{1}{2}$	160.7	43	.30

Underburned Clay Brick

81	Well laid, concentric load.	1-3 port. cement	10-0 $\frac{1}{2}$	$12\frac{1}{2} \times 12\frac{1}{2}$	161.3	40	.46
82	do.	do.	9-11 $\frac{1}{2}$	$12\frac{1}{2} \times 12\frac{1}{2}$	161.9	40	.44

TABLE 8.

DATA OF TESTS OF BRICK COLUMNS.

Col. No.	Maximum Load		Age at Test days	Manner of Failure	Initial Modulus of Elasticity, lb. per sq. in.	Maximum Unit Shortening	Proportional Load at which Popping was First Noted
	Total pounds	Unit, lb. per sq. in.					
Shale Building Brick							
51	507 000	3220	66	Split along joint, top to bottom.	4 350 000	.00104	.59
52	557 000	3510	68	Total collapse with little warning.	5 450 000	.00110	.60
53	527 000	3370	65	do.	4 550 000	.00122	.50
59	601 000	3790	181	do.	4 700 000	.00106	.75
60	651 000	4110	181	Upper half shattered on seventh repetition of load.	5 350 000	.00129	...
55	454 000	2860	69	Total collapse with little warning.	3 550 000	.00102	.66
56	462 000	2980	68	Middle portion thrown out.	3 500 000	.00094	.65
57	427 000	2720	69	Split through joints, near top.	4 100 000		.71
58	452 000	2880	67	Split through joints. Center thrown from machine.	4 700 000		.52
61	350 000	2190	66	Cracked from bottom to top.	3 500 000	.00109	.57
62	358 000	2260	65	Total collapse after longitudinal cracks in top	3 000 000	.00129	.46
91	277 000	1750	66	Spalled at joints. Did not collapse.	800 000	.0027	.40
71	216 000	1360	67	Badly spalled. Mortar reduced to powder. Did not collapse.	101 000		.38
72	248 000	1540	66	Column deflected noticeably to north, spalling and crushing on south. Did not collapse.	107 000		.48
Underburned Clay Brick							
81	166 000	1030	63	Total collapse.	435 000	.00270	.91
82	177 000	1090	62	do.	430 000	.00250	.62

14. *Strength of Brick Columns.*—The results of the tests of brick columns, given in Table 8, page 25, show greater uniformity than has usually been found for test specimens of this kind. A summarized statement of average values from the tests is given in Table 9, page 26. The average value of the crushing strength of brick loaded flatwise was used in computing the ratio in the fourth column of the table. The average value of the unit-loads taken by the three shale brick columns laid up with 1-3 portland cement mortar and tested at 67 days, is used in computing the ratios in the fifth column of the table.

The columns which were built of shale brick, using 1-3 and 1-5 portland cement mortar, and loaded in a similar manner, give

TABLE 9.
SUMMARY OF TESTS OF BRICK COLUMNS.
Average Values.

Ref.	Characteristics of Columns	Average Unit Load, lb. per sq. in.	Ratio of Strength of Column to Strength of Brick	Ratio of Strength of Column to Strength of "A"	Crushing Strength of 6-in. Mortar Cubes, lb. per sq. in.	Ratio of Strength of Column to Strength of Cubes
Shale Building Brick						
A	Well laid, 1-3 portland cement mortar, 67 days.	3365	.31	1.00	2870*	1.17
B	Well laid, 1-3 portland cement mortar, 6 months.	3950	.37	1.18		
C	Well laid, 1-3 portland cement mortar, eccentrically loaded, 68 days.	2800	.26	.83		
D	Poorly laid, 1-3 portland cement mortar, 67 days.	2920	.27	.87	2870*	1.05
E	Well laid, 1-5 portland cement mortar, 65 days.	2225	.21	.66	1710	1.30
F	Well laid, 1-3 natural cement mortar, 67 days.	1750	.16	.52	305	5.75
G	Well laid, 1-2 lime mortar, 66 days.	1450	.14	.43		
Underburned Clay Brick						
H	Well laid, 1-3 portland cement mortar, 63 days.	1060	.27	.31	2870*	.37

*Average value based on 13 tests of 1-3 portland cement mortar cubes 60 days old.

data on the effect of the richness of the mortar in columns tested at the same age. For the columns with 1-3 portland cement mortar 67 days old, the ratio of the breaking load on the columns to the (average) ultimate load carried by a single brick tested flatwise in compression is 0.31; for 1-5 mortar, other conditions being the same, this value is 0.21. These values bear to each other nearly the same ratio as do the strengths of the mortars as shown by the results of tests of 6-in. cubes. This shows a decrease of 34 per cent due to the use of a leaner mortar. For the two columns built of 1-3 portland cement mortar which were tested at an age of 6 months, the value of this ratio is about 0.37, showing an increase of about 21 per cent in the strength of the column during the interval. No mortar cubes were tested at 6 months; but this increase of 21 per cent agrees well with the increase in strength of 1-3 portland cement mortar from 60 days to 6 months, as found in other laboratory tests.

The popping sounds referred to under "13, Phenomena of Tests of Brick Columns" have been accredited to the breaking of the brick in flexure due to the stresses introduced by the readjustment of the different parts following unequal shortening in different parts of the column at a given level, or to uneven bearing of the brick throughout their length, or to both. The practice of bricklayers in placing mortar at the ends of the bricks causes them to be more fully supported at the ends (or not to have a uniform bearing throughout their length). As the same thing is done in the course next above and the joints are broken, the effect is that any given brick has a greater load in the middle and its main support is at the ends. This is particularly true of the inner portion of each joint in a column of small section. It is therefore subject to flexure. After the bricks are broken and vertical cracks formed, this uneven distribution of the pressure and bearing has the effect of eccentricity of loading.

The load at which this popping was first observed is indicated by a cross (x) on the load-deformation diagrams given on pages 30 and 32. This load corresponds to a unit deformation of about 0.00049 for the columns built of 1-3 mortar and tested at 67 days. For similar columns tested at an age of 6 months the value of this unit deformation was somewhat higher, though not so well defined. With the leaner and weaker mortars the value of this deformation is from 0.0017 to 0.0135. For the clay-brick columns

(1-3 portland cement mortar) the deformation is 0.0023 and 0.0026. It seems probable that the popping noises are indications of incipient failure and that this load represents the maximum load which the column would continue to carry even under the conditions present during the test. By reference to the load-deformation diagrams it will be seen that this point coincides closely in nearly every case with the load at which a definite change in the nature of the curve occurs.

The two columns (No. 55 and 56) which were laid up hurriedly (poorly laid) give results which do not differ from those found in the columns built by the regulation methods as much as might have been expected. The average values show a decrease of strength of about 13 per cent due to this cause.

The ratio of the strength of the one column laid up with 1-3 Bricklayers' (natural) cement mortar to the strength of the shale brick columns built of 1-3 portland cement mortar is 0.52. The ratio of the strengths of the corresponding 6-in. cubes as given in Table 9, page 26, is 0.11.

No direct comparison of the strength of the lime-mortar columns with the strength of the mortar can be made (since no tests of the lime mortar were made), although it is evident that the low values observed are due to the low crushing strength of the mortar. From the action of these columns during the tests it seems evident that the lime mortar broke down at a load which was proportionally very much lower than that carried by the cement mortar. From the early signs of distress exhibited by these columns it seems doubtful if they would continue to carry a load greater than about one-third the maximum load given in the test, while for the shale brick columns built with portland cement mortar, the load at which the first signs of distress were observed is from 50 per cent to 75 per cent of the maximum load carried.

For the two columns built of underburned clay brick and 1-3 portland cement mortar, the average load carried was 1060 lb. per sq. in. The ratio of this load to the load taken by the shale brick columns tested at the same age is 0.31. The ratio of the crushing strength of the clay brick to that of the shale brick is 0.37.

It is evident that the strength of any brick or block structure is influenced greatly by the quality of the mortar used. These comparisons indicate that for the columns tested the strength of

columns built from the same brick is closely proportional to the strength of the mortar used in their construction, and the strength of columns built of different brick using the same mortar is closely proportional to the crushing strength of the brick. Of course, these conclusions may not be expected to hold for all combinations of brick and mortar. A mortar might be used of such low crushing strength that the strength of the poorest brick would be great in comparison.

These tests indicate that the crushing strength of the brick is as important a factor in the strength of a structure of this kind as the quality of the mortar used. The most economical structure would seem to result from using a mortar comparable in strength with the brick. Such considerations are generally unnecessary except in design of columns, piers, etc., which are to sustain excessive unit loads.

If, as pointed out in a preceding paragraph, the load at which the column first shows signs of distress by popping sounds represents the maximum permanent load which the column would carry, then this load is the one on which the factor of safety should be based, and not the load carried momentarily before failure.

The effect of eccentric loading and repeated loading on brick columns will be discussed in detail in succeeding paragraphs.

15. *Load-deformation Diagrams for Brick Columns.*—In Fig. 5 and 6, the diagrams show the relation of longitudinal deformation to the applied load for the brick columns tested. The make-up and loading of each column may be found by reference to Table 7. The deformation and load at which the first popping noises were heard are indicated on the diagrams by a cross (x). The curves represent the average of the deformations observed on two opposite faces of the columns. They were plotted from the extensometer measurements and the applied loads. The deformations were measured over a gauged length of about 100 inches.

The curves show the rate at which the column is shortening at any particular load. It is seen that there is not a direct proportionality between the applied load and the resulting deformation, though there is an approach to this for the lower loads. If a straight line be drawn through the points marking the earlier loads, (generally speaking, tangent to the curve), it will represent what would be the modulus of elasticity of the column if it

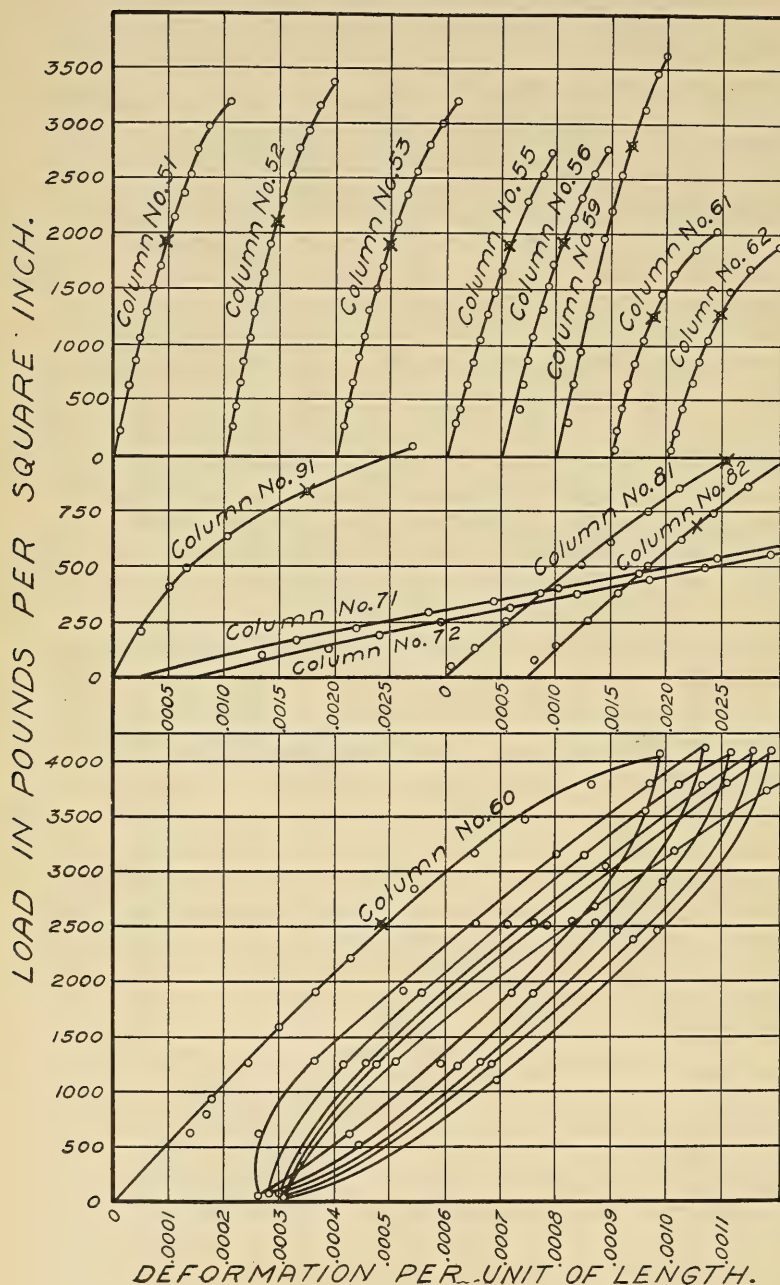


FIG. 5. LOAD-DEFORMATION DIAGRAMS FOR CONCENTRICALLY LOADED BRICK COLUMNS.

continued to compress at the same rate for all loads as it did for the earlier loads. The modulus of elasticity, found from the line so drawn, is termed the "initial modulus of elasticity". The value of this function is given for the brick columns in Table 8, page 25.

The load-deformation curves for the two columns (No. 57 and 58) which were loaded eccentrically are given in Fig. 6, page 32. The maximum deformation is at one face (the face nearer the point of application of the load), and the minimum deformation is at the opposite face. Between the two faces, on the assumption of a plane section before loading remaining a plane section after the load is applied, the deformation will vary uniformly between the values given on the diagram for the corresponding loads on the two faces. The interpretation of the variation of the stresses across the section is given in another place.

16. *Effect of Repeated Loads on Brick Columns.*—One brick column (No. 60, shale brick, 1-3 portland cement mortar, 6 months old) took the full load of the testing machine (651 000 lb., 4110 lb. per sq. in.) without failure. This load was released and reapplied seven times. Upon the seventh application of the load, before the instruments could be read, the column failed with little warning. The failure was complete, as described above for this class of columns. This column is exceptional in the load carried without failure. The corresponding column (No. 59) failed under a single application of a load of 601 000 lb. The stress-deformation diagram for Column No. 60 is given in Fig. 5, page 30. The initial modulus of elasticity for the first application of load is very high (5 400 000 lb. per sq. in.). The value of this function for the succeeding applications is somewhat reduced, being about 4 500 000 lb. per sq. in. The curves for the first application of load are similar to those of other columns of this class. Throughout the repetition, the observed deformations on two opposite faces of the column were nearly the same, showing the absence of bending. The diagram shows the gradual increase in the total deformation under succeeding applications of the load. Upon release of the load there is a "set" which is nearly equal to the increase in total deformation due to the previous application of the load. The curve for decreasing load is seen to be almost the reverse of that for increasing loads (concave upward). This phenomenon has been termed the "inertia of strain", i. e., the lag of

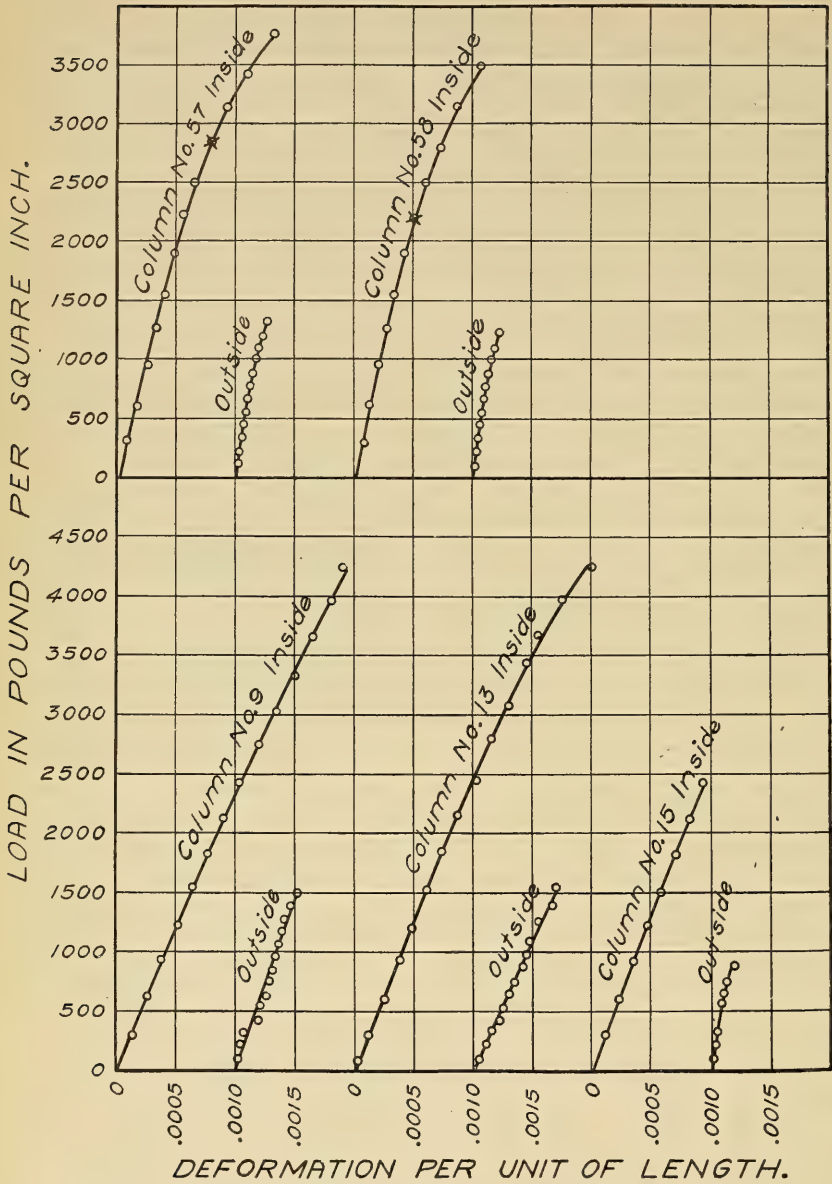


FIG. 6. . LOAD-DEFORMATION DIAGRAMS FOR ECCENTRICALLY LOADED BRICK AND TERRA COTTA BLOCK COLUMNS.

the deformation behind the load. This effect is particularly noticeable at the beginnings and ends of the curves where there is a reversal in the character of the increments of load. It is probable that this column would have taken a load of about 700 000 lb. (4370 lb. per sq. in.) upon a single application to failure. This conclusion is based upon the amount of deformation which probably would have been required to produce failure. It is plain that the release and reapplication of a load produces failure at a lower load than would be necessary at a single application. It seems possible that the indefinite reapplication of the load which produced the first popping sounds mentioned above might finally produce failure. This load corresponds to a load 58 per cent of the load that the column probably would have carried upon a single application.

17. *Effect of Eccentric Loading of Brick Columns.*—In the two brick columns (No. 57 and 58) which were loaded eccentrically, the load was applied at either end of the column along a line 1 in. off the middle of the section, as described under “11. Methods of Testing”. An amount of eccentricity of the load was chosen so that none but compressive stresses might exist in the column. For a rectangular column having the property of proportionality of stress and deformation, in order that tensile stress be not developed, the point of application of the load should fall within the middle third of the section. For such a material as brick masonry it is evident that, as the deformation will not be proportional to the stress at the higher loads, the distribution of the stress across the section may differ somewhat from that assumed in the ordinary analysis. However, this analysis will be used as a basis of comparison with the stresses determined from the observed deformations.

The formula for maximum and minimum stress in a rectangular column subjected to eccentric loading, based upon proportionality of stress and deformation, and neglecting the further eccentricity due to the bending of the column, is

$$f = \frac{P}{A} \left(1 \pm \frac{6\epsilon}{d} \right)$$

where f is the unit-stress at the inner or outer face of the column, respectively, P is the total load on the column, A is the area of the section of the column, ϵ is the eccentricity of loading or distance from the line of application of the load to the center line of

the section, and d is the dimension of the column perpendicular to the line along which the load is applied. For a value of e of 1 inch and a side d of $12\frac{1}{2}$ in., this formula becomes

$$f = \frac{P}{A} (1 \pm 0.48).$$

In other words, the stress at the inner face is 48 per cent greater, and at the outer face it is 48 per cent less, than the average stress over the section of the column. The values of the minimum stress and the maximum stress corresponding to several average stresses are given in Table 10, page 34.

To determine the minimum stress and the maximum stress on the column from the deformations observed at any load, use may be made of the stress-deformation diagram of a column centrally loaded. The stress corresponding to the deformation of the eccentrically loaded column may in this way be taken direct from the diagram for the centrally loaded column. Three columns (No. 51, 52, and 53) are comparable to the two eccentrically loaded columns in materials and fabrication, and their load-deformation diagrams may be considered to give properties representative of the latter two columns. For comparison, the deformations at the inner and outer faces of No. 57 and 58 were taken from their diagrams for several loads, and the unit-stresses correspond-

TABLE 10.

COMPARISON OF STRESSES IN ECCENTRICALLY LOADED BRICK COLUMNS
Stresses are given in pounds per square inch.

Average Unit Stress over Section of Column		Stress at Inner Face of Column		Stress at Outer Face of Column	
		From Deformations	From Formula	From Deformations	From Formula
Column No. 57	500	840	740	245	260
	1000	1590	1480	430	520
	1500	2290	2220	600	780
	2000	2900	2960	780	1040
	2500	3760	3700	1065	1300
Column No. 58	500	702	740	272	260
	1000	1340	1480	492	520
	1500	2105	2220	675	780
	2000	2840	2960	845	1040
	2500	3570	3700	1127	1300

ing to these were found by the use of the load-deformation diagram of each of the three centrally loaded columns named. The average of the three values of the stresses so determined for each case is given in Table 10, page 34, in the columns headed "From Deformations". It will be seen that the values determined from the deformations agree fairly well with the results of the formula, considering the indefiniteness in the amount of eccentricity and the lack of precision in measuring the deformations.

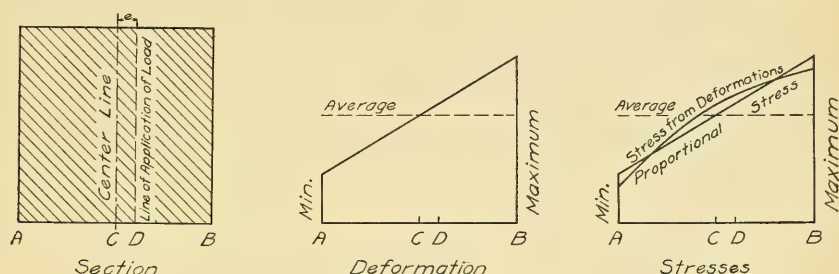


FIG. 7. DISTRIBUTION OF DEFORMATION AND STRESS IN AN ECCENTRICALLY LOADED COLUMN.

The stress at the inner face for the load which produced failure, calculated by the formula given above, is 4030 lb. per sq. in. for No. 57, and 4260 lb. per sq. in. for No. 58. This is considerably greater than the stress on the three centrally loaded columns at failure (average, 3365 lb. per sq. in.). The material under maximum stress is evidently restrained and aided by that near it. It is worth while, also, to call attention to the effect of eccentric loading, since for an eccentricity of 1 inch the columns failed at an average load of 2800 lb. per sq. in., while the centrally loaded ones carried 3365 lb. per sq. in., indicating a loss of 17 per cent in carrying capacity due to the eccentric loading. This result, obtained with a small eccentricity, emphasizes the need of providing for such stresses, or of designing the structure to avoid them so far as possible.

It is interesting to note that although the deformation at the inner face was 67 per cent more than the average, and that at the outer face as much less, the stresses found at the two faces are only about 48 per cent more and less than the average. It may also be of interest to compare the stresses at different points across the column.

In Fig. 7, page 35, the middle diagram shows the distribution of the deformation across the column, assumed to vary uniformly. The diagram on the right gives the distribution of stresses. The straight line in the latter stress-deformation diagram represents the assumptions of the ordinary analysis. The curved line gives the stresses determined from the deformations by the method used in finding the stresses at the outer and inner faces given in Table 10, page 34.

B. TERRA COTTA BLOCK COLUMN TESTS.

18. *Terra Cotta Block Column Tests.*—Sixteen terra cotta block columns were tested. Table 11, page 39, gives data on the dimensions, make-up, and method of loading the columns. The columns were built and tested in two lots, an interval of about one year separating the times of making the tests. The two lots of columns were built of blocks which came in different shipments. The cement used was the same brand in both years, though the lots were different. The terra cotta block columns were generally made in sets of two. Each set was constructed and loaded similarly. Three of the columns were laid up hurriedly (poorly laid); the remainder were built with the usual care given to such work, as described on page 17. The number of variables in these tests was smaller than was the case in the tests of brick columns, but the variation in materials makes comparisons of results more difficult. The load was applied to the columns in different ways. Generally the load was applied continuously to failure. In two cases (No. 7 and No. 8) the maximum load that could be applied with the machine was repeated several times without loading the columns to failure. No. 7 was loaded eccentrically also but not enough load was applied in this way to cause failure. These two columns were removed from the testing machine and have been set aside for future tests. Three columns were loaded eccentrically to failure. Two columns (No. 6 and No. 7) had an increasing load applied and released in succession. This method of loading will be understood by reference to the stress-deformation diagrams for these columns, Fig. 10, page 46.

19. *Phenomena of Tests of Terra Cotta Block Columns.*—The terra cotta block column tests resembled the tests of brick columns in many respects. In the following paragraphs it will fre-

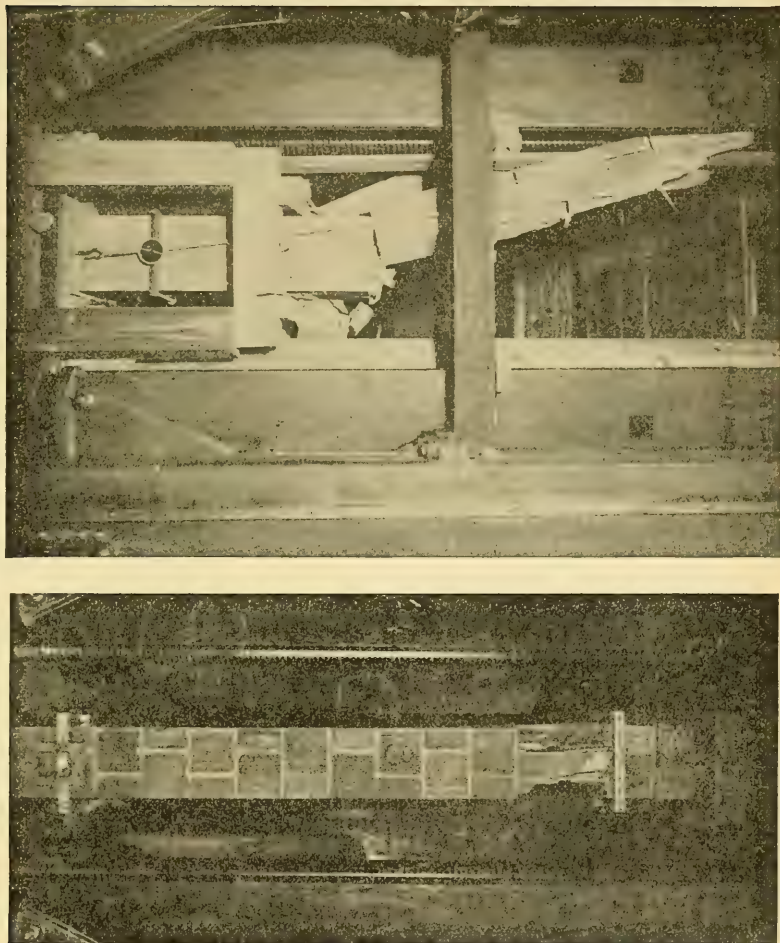


FIG. 8. VIEWS SHOWING MANNER OF FAILURE OF TERRA COTTA BLOCK COLUMNS NO. 4 AND NO. 18.

quently be convenient to refer to the parallel description of the tests of brick columns to avoid repetition.

Generally the terra cotta block columns gave no sign of distress until a load near the maximum was applied, when cracking noises, similar to those described for the brick columns, were heard. This was soon followed by the formation of longitudinal cracks through the vertical joints or the spalling of the horizontal mortar joints at the corners of the column, either of which was immediately followed by sudden collapse of the column. The failures of these columns were even more violent than those described for the brick columns. The notes for test of Column No. 14 state that there were 18 whole blocks found in the debris after the collapse of the column; over 70 per cent of the blocks were broken. The percentage of breakage was not so great in the columns of smaller section, since failure in these was more local. The smaller columns showed more variation in the manner of failure than the large ones. The characteristic form of failure for the $12\frac{1}{2} \times 12\frac{1}{2}$ in. columns was a sudden total collapse immediately following the appearance of longitudinal cracks through the vertical joints near the middle. Failure sometimes occurred with no warning except the continued shortening of the column under the increasing load. The photographs (Fig. 8 page 37) show failures of some of these columns which are characteristic.

The columns loaded eccentrically failed by splitting from end to end along the vertical mortar joint which was parallel to and nearer the loading bar. In the test of No. 15 under eccentric load, after having been loaded centrally, failure came prematurely by the breaking of the lower bearing block due to the excessive cross-breaking stress arising from this method of applying the load to the column. The load at failure can not be considered to be the strength of this column even under eccentric loading.

The fractured surfaces of No. 17, (built of apparently inferior blocks) showed many light-colored interiors. This appearance confirms the judgment that was exercised in selecting these as underburned blocks.

20. *Strength of Terra Cotta Block Columns.*—The results of the test of terra cotta block columns are given in Table 12, page 40. A summary of average values from the tests is given in Table 13, page 42. The variation in materials and the method of applying the load, together with the inability to load some of

TABLE 11.

DATA OF TERRA COTTA BLOCK COLUMNS.

Col. No.	Characteristics of Column and Loading	Kind of Mortar	Length ft.-in.	Nominal Section in. x in.	Gross Area sq. in.	Number of Courses	Average Thickness of Joints inches
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Columns tested January, 1907.

1	Well laid, loaded concentrically	1-2 portland	12-6	$8\frac{1}{2} \times 8\frac{1}{2}$	72.8	18	.33
2	do.	do.	12-6 $\frac{1}{2}$	$8\frac{1}{2} \times 8\frac{1}{2}$	72.8	18	.35
3	do.	do.	11-11	$8\frac{1}{2} \times 13$	110.5	17	.41
4	do.	do.	11-10 $\frac{1}{2}$	$8\frac{1}{2} \times 13$	112.1	17	.38
5	do.	do.	12-7 $\frac{1}{4}$	13×13	170.5	18	.41
6	do.	do.	12-7	13×13	169.0	18	.39
7	Loaded concentrically and eccentrically	do.	12-7 $\frac{3}{4}$	$17\frac{1}{2} \times 17\frac{1}{2}$	307.3	18	.42

Columns tested January, 1908.

8	Well laid, loaded concentrically	1-3 portland	9-9	$12\frac{1}{2} \times 12\frac{1}{2}$	162.6	14	.34
15*	do.	do.	9-9	$12\frac{1}{2} \times 12\frac{1}{2}$	162.6	14	.34
9	Well laid, loaded eccentrically	do.	9-9	$12\frac{1}{2} \times 12\frac{1}{2}$	162.6	14	.36
14	Poorly laid, loaded concentrically	do.	9-9	$12\frac{1}{2} \times 12\frac{1}{2}$	161.3	14	.36
13	Poorly laid, loaded eccentrically	do.	9-9	$12\frac{1}{2} \times 12\frac{1}{2}$	161.3	14	.36
12	Poorly laid, loaded concentrically	do.	9-9	$12\frac{1}{2} \times 12\frac{1}{2}$	162.5	14	.36
17	Well laid with inferior block, concentric load	do.	9-9 $\frac{1}{2}$	$12\frac{1}{2} \times 12\frac{1}{2}$	163.8	14	.39
18	Well laid, loaded concentrically	1-5 portland	9-10	$12\frac{1}{2} \times 12\frac{1}{2}$	164.3	14	.43
19	do.	do.	9-10	$12\frac{1}{2} \times 12\frac{1}{2}$	161.3	14	.43

*Later loaded eccentrically to failure.

TABLE 12.

DATA OF TESTS OF TERRA COTTA BLOCK COLUMNS

Col. No.	Maximum Total Load pounds	Unit Load lb. per sq. in.	Age at Test days	Initial Modulus of Elasticity lb. per sq. in.	Maximum Unit Shortening
Columns tested January, 1907.					
1	219 000	3030	68	2 170 000	.00160
2	200 000	2740	66	1 910 000	.00153
3	300 000	2700	67	No readings taken	
4	388 100	3440	68	2 150 000	.00175
5	543 400	3200	65	2 040 000	.00200
6	458 000	2710	68	2 700 000	.00142
7	607 700+	1980+	69	2 200 000	
Columns tested January, 1908					
8	606 000+	3730+	68	2 780 000	.00154
15	616 000+	3790+	63	2 750 000	*.00170
9	565 000	3470	64	2 330 000	
14	538 000	3330	63	2 860 000	.00134
13	501 000	3110	67	2 500 000	
12	534 000	3280	69	3 200 000	.00134
17	500 000	3050	68	2 300 000	.00196
18	526 000	3200	66	2 680 000	.00150
19	564 000	3500	64	2 700 000	.00148

*Based upon the deformation produced by the first application of a central load.

The loads in table followed by a plus (+) sign are not the ultimate loads of the columns, but are the maximum loads that could be applied with the testing machine used.

the columns to failure as planned, make comparisons of results uncertain in some cases.

It will be seen by comparison of the results of the tests made January, 1907, that there is no appreciable difference in the strength of the columns due to a variation in the size of the section. It was anticipated that column No. 7, $17\frac{1}{2} \times 17\frac{1}{2}$ in. in section, could not be loaded to failure, but it is of interest to know

NOTES ON MANNER OF FAILURE OF TERRA COTTA BLOCK COLUMNS

Col. No.	Manner of Failure
Columns tested January, 1907	
1	Three top courses shattered.
2	Failure followed appearance of crack in 7th and 8th courses from bottom.
3	Compression failure at middle on west side.
4	Crushed at top. Broke in middle.
5	Cracked at top.
6	Horizontal joint spalled in 12th course. Top half of column collapsed.
7	No sign of failure at full capacity of machine. Set aside for future test.
Columns tested January, 1908	
8	Did not fail. Set aside for future test.
15	Did not fail. Later loaded eccentrically.
9	Split end to end along joint.
14	Shattered suddenly, following appearance of vertical cracks near middle.
13	Split end to end along joint.
12	Shattered suddenly, following appearance of vertical cracks near middle.
17	Completely shattered, following formation of vertical cracks near middle.
18	do.
19	do.

that the elastic properties of a column of this size (as shown by the load-deformation curve) are similar to those of smaller columns. The ratio of the maximum loads carried by the 1907 columns which were tested to failure to the crushing strength of individual blocks is about 0.86.

No tests were made of the mortar used in Columns No. 1 to 7, inclusive. The tensile tests of 1-3 mortars, given in Table 5, page 15, indicate that the two lots of Universal portland cement were similar. From this we may conclude that the rich mortar used in the 1907 columns was proportionally stronger than the

mortar used in the 1908 columns. The great difference in the strengths of the columns built from the two shipments of blocks seems to be due to the form of the blocks themselves. As was pointed out in a previous paragraph, the blocks in the first lot were cut with their ends (bearing faces) concave in the form of a

TABLE 13.

SUMMARY OF TESTS OF TERRA COTTA BLOCK COLUMNS

Average Values

Ref.	Characteristics of Columns	Numbers of Columns Used in Average	Average Maximum Unit Load lb. per sq. in.	Ratio of Strength of Column to Strength of Block (Gross area)	Crushing Strength of 6-in. Mortar Cubes lb. per sq. in.	Ratio of Strength of Column to Strength of Cubes	Ratio of Strength to Estimated Strength of "E"
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Columns tested January, 1907

1-2 portland cement mortar. All well laid and centrally loaded.

A	8½ x 8½ in.	1, 2	2885	.83			
B	8½ x 13 in.	3, 4	3070	.89			
C	13 x 13 in.	5, 6	2955	.85			
D	17½ x 17½ in.	7,	1980+				

Columns tested January, 1908

12½ x 12½ in., 1-3 portland cement mortar, well laid unless noted.

E	Central load	8, 15,	3790+ 4300*	.74+ .83*	3400	1.12+ 1.26*	 1.00*
F	Eccentric load	9,	3470	.65	3090	1.12	.81*
G	Poorly laid, central load	12,	3305	.64	3130	1.05	.76
H	Poorly laid, eccentric load	13,	3110	.60	3025	1.06	.75
I	Inferior blocks, central load	17,	3050	.59	3370	.88	.71
K	1-5 mortar, central load	18, 19,	3350	.65			.78

*Estimated.

The average age of the columns when tested was 67 days.

cylindrical surface. The variation from a plane was in some cases as much as $\frac{3}{16}$ in. The blocks from the second shipment had end faces more nearly plane. The variation in strength of these blocks as found from the tests of individual blocks as well as in the column tests seems to be due to the form of the bearing faces. Other comparisons of the two lots of blocks show them to be similar, both as to material and manufacture. The blocks from which the 1907 columns were made, failed in the compression test at a lower load apparently on account of the non-uniform distribution of the load over the section. The blocks crushed first at the high points of the ends, and complete rupture soon followed. Fig. 9, page 43, shows the conditions which seem to

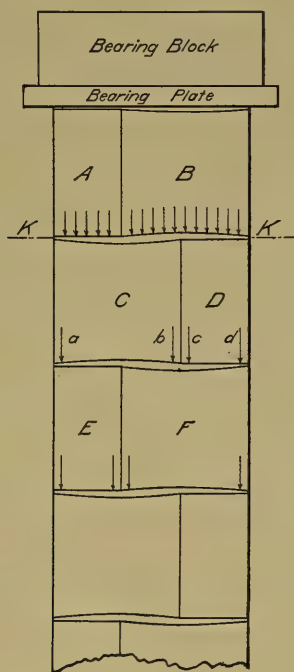


FIG. 9. DISTRIBUTION OF PRESSURE IN A COLUMN BUILT OF BLOCKS WITH CURVED ENDS.

exist in a column built of the 1907 blocks. The amount of curvature in the faces of the blocks has been somewhat exaggerated in this sketch. Some course of blocks near the top as A and B, may be assumed to receive a uniformly distributed load and

transmit it as such to the joint below. The curvature of the ends of the blocks gives a variable thickness to the joint below, and on account of the greater amount of yielding in the thicker part of the joint, the block D will deliver its load somewhat concentrated at d. Similarly, the pressure transmitted downward by C will be greater at a than at b or at points between. This change in the distribution of the pressure will produce a horizontal force tending to open the vertical cracks. Likewise any concentration of the load on any block F near its middle accompanied by a concentration of the supporting pressure below at points near its ends will tend to produce cross breaking along a vertical line. It is clear that the conditions described would become intensified as the load is transmitted to each lower course, until the middle of the column is reached. This explanation of the distribution of the stresses may account for some of the phenomena of these tests. If the blocks safely resisted the stresses tending to split them, there remained the danger of crushing the mortar at the corners due to excessive pressure existing there. The columns built from 1907 blocks failed by splitting the blocks through a longitudinal joint or by crushing the mortar at the corners. Under ordinary conditions the mortar should carry a larger load than came on these columns.

None of the columns well laid, using 1908 blocks and 1-3 mortar, failed by applying a central load up to the capacity of the testing machine. It is estimated from the character of the load-deformation curves for Columns No. 8 and No. 15 and from the strength of the blocks, that these columns would have required a load of about 4300 lb. per sq. in. to cause failure. The strength of similar columns under eccentric load also points to the same conclusion.

Column No. 12, which was laid up hurriedly, gave results relatively much lower than have been found in the tests of brick columns under similar conditions. Similar columns (No. 13 and No. 14) which were loaded eccentrically gave values nearly as large as No. 12. The strengths of the poorly-laid columns under eccentric load seem to be more nearly representative than the centrally loaded one.

The column (No. 17) laid up with inferior blocks gave values about 71 per cent of the estimated strength of the similar columns built of better blocks.

The two columns (No. 18 and No. 19) built of 1-5 portland cement mortar gave results about 22 per cent lower than the estimated values for the 1-3 mortar columns.

21. *Load-deformation Diagrams for Terra Cotta Block Columns.*—Longitudinal deformations were measured on all the terra cotta block columns except No. 3. The load-deformation curves have been plotted for each column in the same manner as was described for the brick columns. In some cases where the same column

TABLE 14.

COMPARISON OF STRESSES IN ECCENTRICALLY LOADED TERRA COTTA BLOCK COLUMNS.

Stresses are given in pounds per square inch.

Average Unit Stress over Section of Column		Stresses at Inner Face of Column		Stresses at Outer Face of Column	
		From Deformations	From Formula	From Deformations	From Formula
Column No. 7	500	1125	1010	200	—12
	1000	1940	2025	315	—25
Column No. 9	500	718	735	252	265
	1000	1340	1470	487	530
	1500	1930	2210	603	795
	2000	2530	2940		1060
Column No. 13	500	900	735	350	265
	1000	1700	1470	680	530
	1500	2350	2210	1060	795
	2000	2950	2940	1400	1060
	2500		3680	1700	1225
Column No. 15	500	750	735	100	265
	1000	1500	1470	250	530
	1500	2500	2210	400	795

Column No. 7. $d = 17.55$ in. $e = 3$ in.

Stresses from deformations are those corresponding to the first application of a previous central load on the same column. Stresses from formula are about 100% more or less than the average stress across the section.

Column No. 9. $d = 12.75$ in. $e = 1$ in.

Stresses from deformations were computed from deformations of Columns No. 8 and 15, for first application of central load. Stresses from formula are 47% more or less than the average.

Column No. 13. $d = 12.75$ in. $e = 1$ in.

Stresses from deformations were computed from deformations of Columns No. 12 and 14.

Column No. 15. $d = 12.75$ in. $e = 1$ in.

Stresses from deformations were computed from deformations for first application of central load on Columns No. 8 and 15.

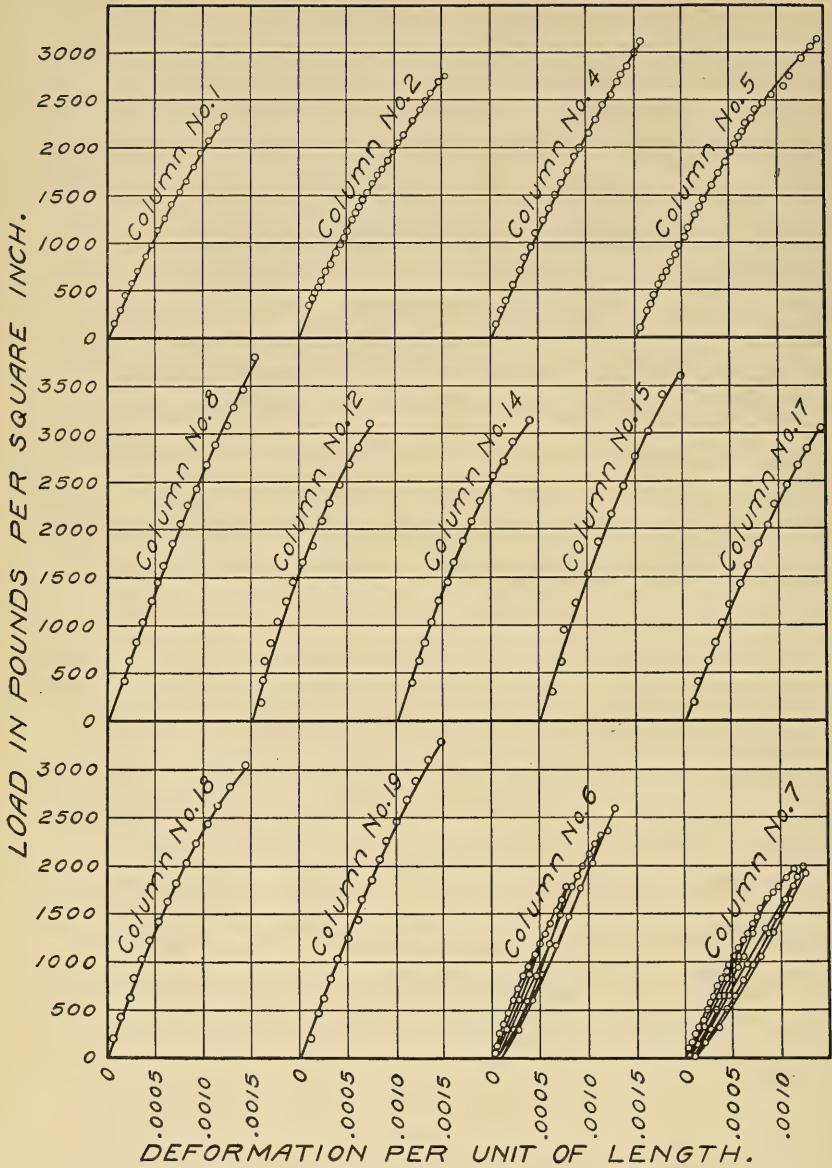


FIG. 10. LOAD-DEFORMATION DIAGRAMS FOR CONCENTRICALLY LOADED TERRA COTTA BLOCK COLUMNS.

was loaded in different ways two diagrams are given. In cases of this kind the form of the curve itself is sufficient to indicate the nature of loading. Load-deformation diagrams for the terra cotta block columns are given in Fig. 6, page 32, and Fig. 10, page 46.

22. *Effect of Eccentric Loading of Terra Cotta Block Columns.*—Four of the terra cotta block columns were loaded eccentrically. The amount of the eccentricity was 1 in. in each case, except Column No. 7, where $e = 3$ in. Table 14 page 45, gives a comparison of the stresses across the section of the column as determined by two methods of computation. This table was prepared in the way described for Table 10 for the brick columns. The notes following the table give in detail the origin of the values for each column.

Column No. 7 was not loaded to failure. No. 15 failed prematurely under eccentric load due to the breaking of the bearing block on top of the column and the resulting concentration of the load. This accounts for the smaller number of stresses given in the comparisons for these columns in Table 14.

The terra cotta block columns under eccentric load exhibited phenomena very similar to those observed for the brick columns. The stresses computed from deformations agree fairly closely in each case with those derived from theoretical considerations based on the measured amount of the eccentricity of the load.

23. *Effect of Repeated Loads on Terra Cotta Block Columns.*—On columns No. 7, 8, and 15, the maximum load that could be applied with the testing machine was applied and released a number of times. In no case was failure produced by a re-application of the load, although it seems evident from the increasing amount of deformation produced by the maximum load and the gradually increasing permanent set upon release of the load on Column No. 15 that only a few more applications of this load would have been necessary to produce failure. The maximum deformation produced in Column No. 15 (load-deformation diagram for repeated load not given) by the re-application of the load was 0.00168. The great increase of deformation due to the re-application of these high loads is convincing evidence of the weakening effect of such loading. This column was later loaded eccentrically, but its premature failure due to the breaking of the top bearing block prevented any independent estimate of its original

strength. The similarity in the elastic properties of Columns No. 8 and No. 15 indicates that they probably would have carried nearly the same loads at failure. The comparatively low unit loads which were reapplied to Column No. 7 produced a correspondingly increased total deformation and appreciable set upon release of the load.

C. COMPARISON OF RESULTS.

24. *Summary.*—Both brick columns and terra cotta block columns gave high strengths in all cases where strong mortar and care in building were used. For central loading, the strength of the brick columns ranged from 3220 to 4110 lb. per sq. in., and the strength of the terra cotta block columns from 2700 to 3790 lb. per sq. in., the columns having the highest resistance not failing at the full capacity of the machine. The effect of the strength of the mortar is apparent in the carrying capacity developed in the columns; lower loads were found in columns built with 1-5 portland cement mortar than in those with 1-3 portland cement mortar, still lower loads in those with 1-3 natural cement mortar, and still lower loads in those having 1-2 lime mortar. The effect of the quality of the brick is shown in the columns made with inferior brick, which carried only 31 per cent as much as columns built with the better grade of brick. In the case of the terra cotta columns, the blocks which were culled out as somewhat inferior gave a column strength perhaps 30 per cent less than the columns built with superior blocks. The effect of the attempt to represent hurried or careless workmanship in two brick columns and in three terra cotta block columns was a loss in strength of about 15 per cent and 25 per cent, respectively.

The ratio of the strength of the columns to the compressive strength of the individual brick and block is of interest. In the well-built brick columns loaded centrally, the ratio of strength of column to compressive strength of individual brick ranged from 0.31 to 0.37, and in the underburned clay brick column the ratio was 0.27. In the terra cotta block columns with central loading the ratio of strength of column to that of individual block was 0.74 for the incompletest test and 0.83, 0.85, and 0.89 for the others. If, as seems to be the case, the strength of the brick or block to resist cross-breaking is an element in determining the

strength of the built-up column, a deeper or thicker brick would give higher column strength. It is possible that this partially accounts for the fact that the ratio is found to be higher for terra cotta block columns than for brick columns. The tests suggest that the ability of individual pieces to resist transverse strength is an important element in the strength of the completed column. This suggestion may have an important bearing on the advantageous size of the component blocks which may be used in a compression piece where high strength is desired.

The strength of the column is greater than the strength of the mortar cubes in both brick and terra cotta block columns, excepting only the soft brick columns which had brick of low compressive strength. It is evident that the strength of individual brick or blocks and the strength of the mortar both enter into the resistance of the column. The relative effect of the two depends upon the character of the material. It is evident, however, that the better the individual piece the more important it is to have a mortar of high resisting strength.

The results obtained in applying the load eccentrically were found to agree very well with those obtained from ordinary analysis. When the amount of eccentricity in the application of the load is known or may be estimated closely, the ability of the column to resist this action may be calculated quite closely. It is apparent from the results that the calculated resisting stress in the column on the side of maximum compression is higher than that which causes failure in centrally loaded columns. The higher stress developed with eccentric loading is probably due to the influence of the restraint of the less stressed interior portion. The tests made by applying and releasing a single load a number of times gave failures at loads below those which produced failure in similar columns at a single application of the load. The phenomenon is common in materials of the nature of brick and terra cotta.

It is apparent that the quality of workmanship in laying up such columns has an important bearing upon the resisting strength. The work of building columns, however, is not difficult and requires only ordinary care. Full joints and an even bearing are important, and the ordinary workman ought to be able to construct columns of high strength. In the tests made on columns

intended to represent poor or careless workmanship, the decrease in strength was not as much as anticipated. However, it must be understood that careful and trustworthy work is essential and that a few poor joints will materially reduce the strength of the structure. Wherever good material and good workmanship are insured the strength of masonry of this kind may be utilized with advantage.

PUBLICATIONS OF THE ENGINEERING EXPERIMENT STATION

- Bulletin No. 1.* Tests of Reinforced Concrete Beams, by Arthur N. Talbot. 1904. (*Out of print.*)
- Circular No. 1.* High-Speed Tool Steels, by L. P. Breckenridge. 1905.
- Bulletin No. 2.* Tests of High-Speed Tool Steels on Cast Iron, by L. P. Breckenridge and Henry B. Dirks. 1905.
- Circular No. 2.* Drainage of Earth Roads, by Ira O. Baker. 1906. (*Out of print.*)
- Bulletin No. 3.* The Engineering Experiment Station of the University of Illinois, by L. P. Breckenridge. 1906. (*Out of print.*)
- Bulletin No. 4.* Tests of Reinforced Concrete Beams, Series of 1905, by Arthur N. Talbot. 1906.
- Bulletin No. 5.* Resistance of Tubes to Collapse, by Albert P. Carman. 1906. (*Out of print.*)
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- Bulletin No. 8.* Tests of Concrete: I. Shear; II. Bond, by Arthur N. Talbot. 1906. (*Out of print.*)
- Bulletin No. 9.* An Extension of the Dewey Decimal System of Classification Applied to the Engineering Industries, by L. P. Breckenridge and G. A. Goodenough. 1906.
- Bulletin No. 10.* Tests of Concrete and Reinforced Concrete Columns, Series of 1906, by Arthur N. Talbot. 1907. (*Out of print.*)
- Bulletin No. 11.* The Effect of Scale on the Transmission of Heat through Locomotive Boiler Tubes, by Edward C. Schmidt and John M. Snodgrass. 1907. (*Out of print.*)
- Bulletin No. 12.* Tests of Reinforced Concrete T-beams, Series of 1906, by Arthur N. Talbot. 1907. (*Out of print.*)
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- Bulletin No. 26.* High Steam Pressures in Locomotive Service. A review of a Report to the Carnegie Institution of Washington. By W. F. M. Goss. 1908.
- Bulletin No. 27.* Tests of Brick Columns and Terra Cotta Block Columns, by Arthur N. Talbot and Duff A. Abrams. 1909.

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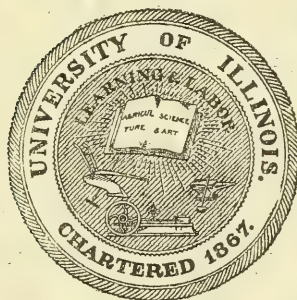
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BULLETIN NO. 28

A TEST OF THREE LARGE REINFORCED CONCRETE BEAMS

BY

ARTHUR N. TALBOT



UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

URBANA, ILLINOIS
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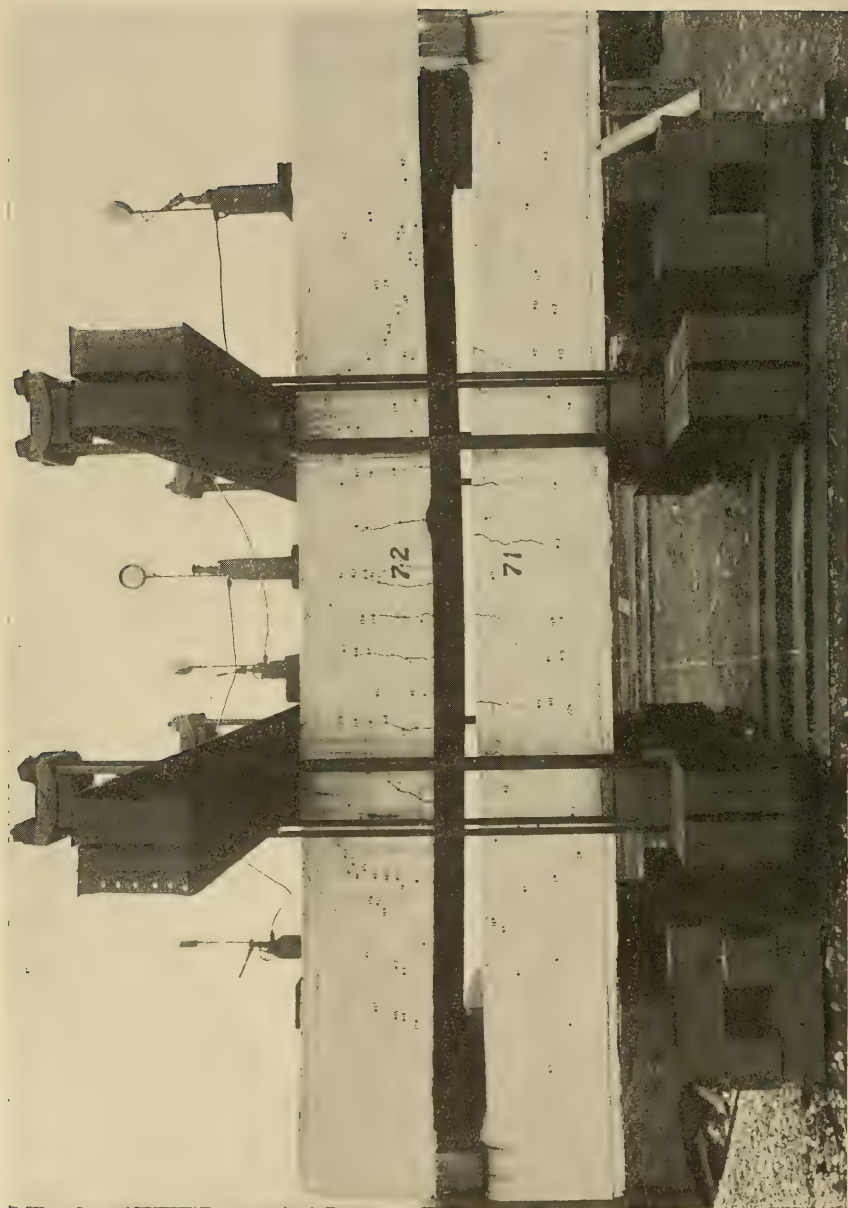
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The control of the Engineering Experiment Station is vested in the heads of the several departments of the College of Engineering. These constitute the Station Staff, and with the Director, determine the character of the investigations to be undertaken. The work is carried on under the supervision of the Staff; sometimes by a research fellow as graduate work, sometimes by a member of the instructional force of the College of Engineering, but more frequently by an investigator belonging to the Station corps.

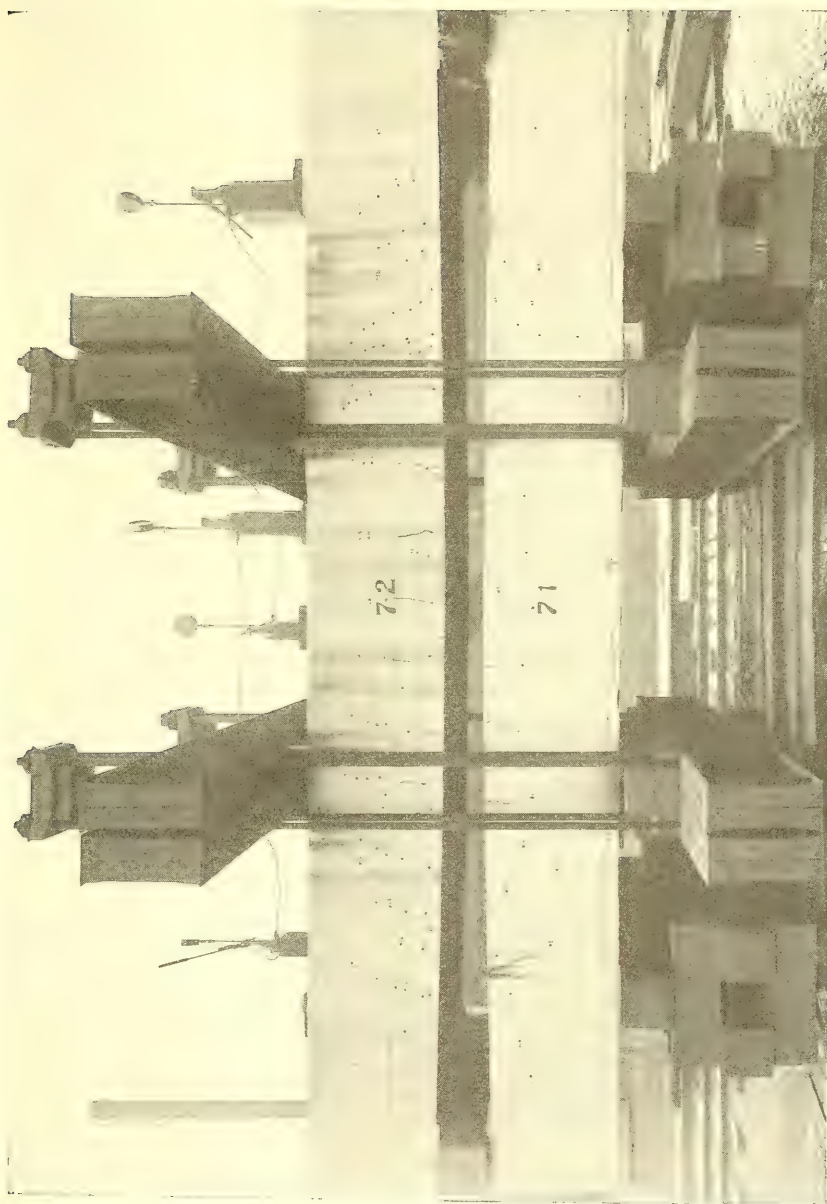
The results of these investigations are published in the form of bulletins, which record mostly the experiments of the Station's own staff of investigators. There will also be issued from time to time in the form of circulars, compilations giving the results of the experiments of engineers, industrial works, technical institutions, and governmental testing departments.

The volume and number at the top of the title page of the cover are merely arbitrary numbers and refer to the general publications of the University of Illinois; *above the title is given the number of the Engineering Experiment Station bulletin or circular, which should be used in referring to these publications.*

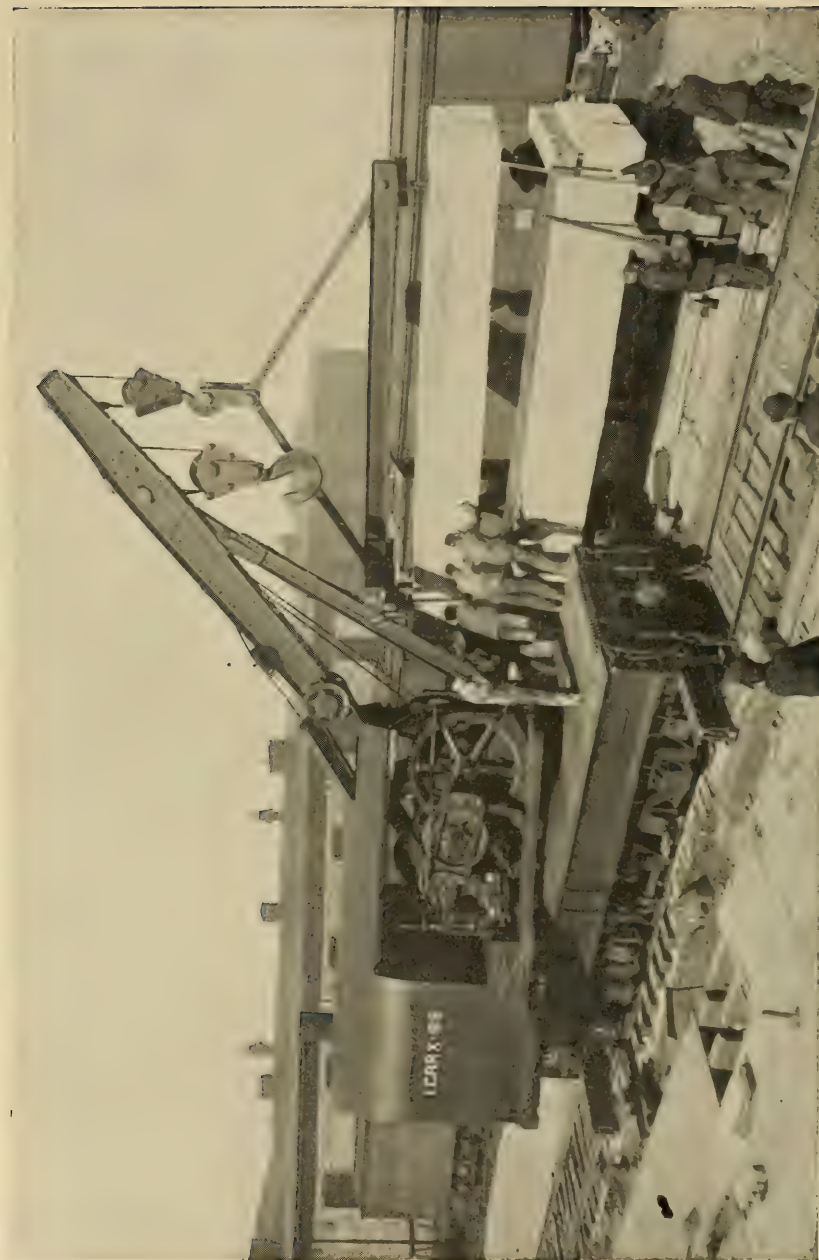
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VIEW SHOWING WEST SIDE AFTER MAXIMUM LOAD.



VIEW SHOWING EAST SIDE AFTER MAXIMUM LOAD.



100-TON DERRICK CAR SETTING TOP SLAB.

UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

BULLETIN No. 28

OCTOBER 1908

A TEST OF THREE LARGE REINFORCED CONCRETE
BEAMS

BY ARTHUR N. TALBOT, PROFESSOR OF MUNICIPAL AND SANITARY
ENGINEERING AND IN CHARGE OF THEORETICAL
AND APPLIED MECHANICS.

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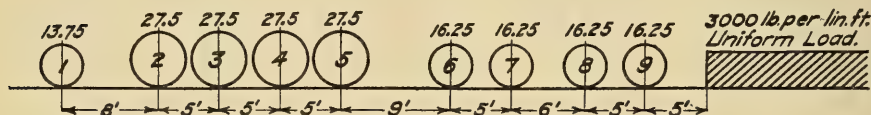
I. INTRODUCTION.

1. *Preliminary.*—It is thought that the tests here described will be of interest because the reinforced concrete beams were chosen almost at random from a large number of beams made up for use in a railroad structure and thus may be considered to be representative of actual conditions of construction, and because the tests give a comparison of the efficiency of two methods of placing the reinforcement to resist diagonal tension or so-called shear failure. In the minds of some engineers test beams made up in laboratories differ so much in make-up and properties from the work found in practical reinforced concrete construction that the results and the conclusions drawn therefrom are not applicable to actual construction. Conservative engineers may have felt that there is some basis for doubt in massive construction, since the difference in size is considerable and since the methods of mixing and fabrication are quite unlike those used in laboratory work. The construction of large reinforced concrete floor slabs for use in the Grand Crossing track elevation construction of the Illinois Central Railroad, perhaps one of the most important pieces of reinforced concrete construction of the kind yet undertaken, gave an opportunity to get results of value; and the tests were undertaken through the co-operation of the Engineering Experiment Station of the University of Illinois and the Department of Bridges and Buildings of the Illinois Central Railroad with a view of determining the properties of large beams made under practical conditions of construction. The beams were perhaps the largest reinforced concrete beams yet tested, and the testing apparatus used and the method of making the test involved some novel features.

The beams were in the form of slabs, 25 ft. long, 6 ft. 3 in. wide, and 34 in. deep at the middle, and weighed about 33 tons each. They were built for use in the bridge floor over subways. The slabs span the distance from the curb to the center of the street pavement (shorter and smaller slabs spanning the sidewalk space) making a floor over the street upon which the ballast is spread and the track laid for the eight track railroad. The slabs were built in the yards of the Illinois Central Railroad near Twenty-seventh St., Chicago, and were to be left where made until ready for transportation to their destination, some seven miles away.

119 of these slabs were made and also 109 smaller ones. The test was made close to the spot where the slabs were built.

2. *Loading.*—In the design of the slabs the Illinois Central Railroad used their standard typical loading of two 188.75-ton engines followed by a uniform load of 6000 lb. per lin. ft. of track, with tracks 12 ft. 6 in. center to center. The design also contemplates the placing of tracks in any position on the slabs laterally. The tracks cross the slabs on a skew, the angle with the axis of the slab being $10^{\circ} 26'$. The distance from top of the slabs to base of rail was first planned to be 12 inches over the thicker portion of the slab, but later this was increased somewhat. The bearing



Loads are given in 1000-lb. units for one rail.

FIG. 1. TYPICAL ENGINE LOADING.

of the slab on the supporting cross beams at either end is 1 ft. 6 in., so that the distance from center to center of supports is 23 ft. 6 in. The maximum bending moment and the maximum shear will be developed by the driver wheel loads. Fig. 1 gives the loads and spacing of the typical engine. Although no exact comparison may be made, computation will show that a test load placed at the one-third points of the span length will develop a bending moment somewhat greater than the same load placed at the driver spacing, and that, although the shear developed as one driver approaches the support may be somewhat greater than that found in the method of the test, the greater diagonal tension resistance in a reinforced concrete beam close to the supports as known from other tests, indicates that the one-third point loading is as severe or more severe a test for diagonal tension resistance than will be found with the engine spacing of the load. It was therefore concluded that the usual test method of placing the load equally at the one-third points of the span length would give results comparable with the distribution of the load which the slab is designed to receive.

3. *Acknowledgment.*—Mr. A. S. Baldwin is Chief Engineer of the Illinois Central Railroad. Mr. R. E. Gaut is Engineer of

Bridges and Buildings, and Mr. F. L. Thompson is Assistant Engineer of Bridges and is in charge of the Grand Crossing track elevation work. To them, and to Mr. C. Chandler, Chief Draftsman in the Department of Bridges and Buildings, acknowledgment is due for facilities afforded and assistance and suggestions given in carrying out the tests. Credit is also due Messrs. D. A. Abrams and W. R. Robinson of the University of Illinois Engineering Experiment Station for efficient aid in the work.

II. SLABS AND METHODS OF TESTING.

4. *Materials.*—The information and data on the materials used and the methods of fabrication were furnished by Mr. Gaut of the Illinois Central Railroad, and as there was no thought of needing results for test discussion, no special measurements or analyses were made, only the attention usual in well-made construction work being given. The sand was torpedo sand of good quality from Merom, Indiana. The stone was unscreened crushed limestone from Chicago, the largest size being two inches. The cement was Owl portland cement manufactured by the German-American Portland Cement Co. Tests of the cement gave a tensile strength of 822 lb. per sq. in. for the 7-day neat test; 95.2 per cent passed the No. 100 sieve.

The main longitudinal reinforcing bars were 1-in. square corrugated bars, new style. Tests made by Robert W. Hunt & Co. of the bars used in this work show an elastic limit varying between 51 260 and 52 770 lb. per sq. in. The $\frac{1}{2}$ -in. bars used in the top longitudinal reinforcement were cup bars. Tests showed these to have an elastic limit ranging from 45 800 to 51 100 lb. per sq. in. The stirrups were $\frac{1}{2}$ -in. cup bars.

5. *The Slab and its Reinforcement.*—The general size of the slab was 6 ft. 3 in. breadth by 25 ft. length. One-half of the length had a general thickness of 34 in., and the other one-half sloped from the middle to a thickness of 31 in. at the end. This difference in thickness was for the purpose of facilitating drainage. In addition to the thickness just mentioned there was an additional inch at the bottom at either end for a distance of 18 in., the width of the bearing area. This arrangement permits a better appearing joint at the supports. Fig. 2 gives the dimensions of the slabs. The estimated weight of one of the slabs is 33 tons.

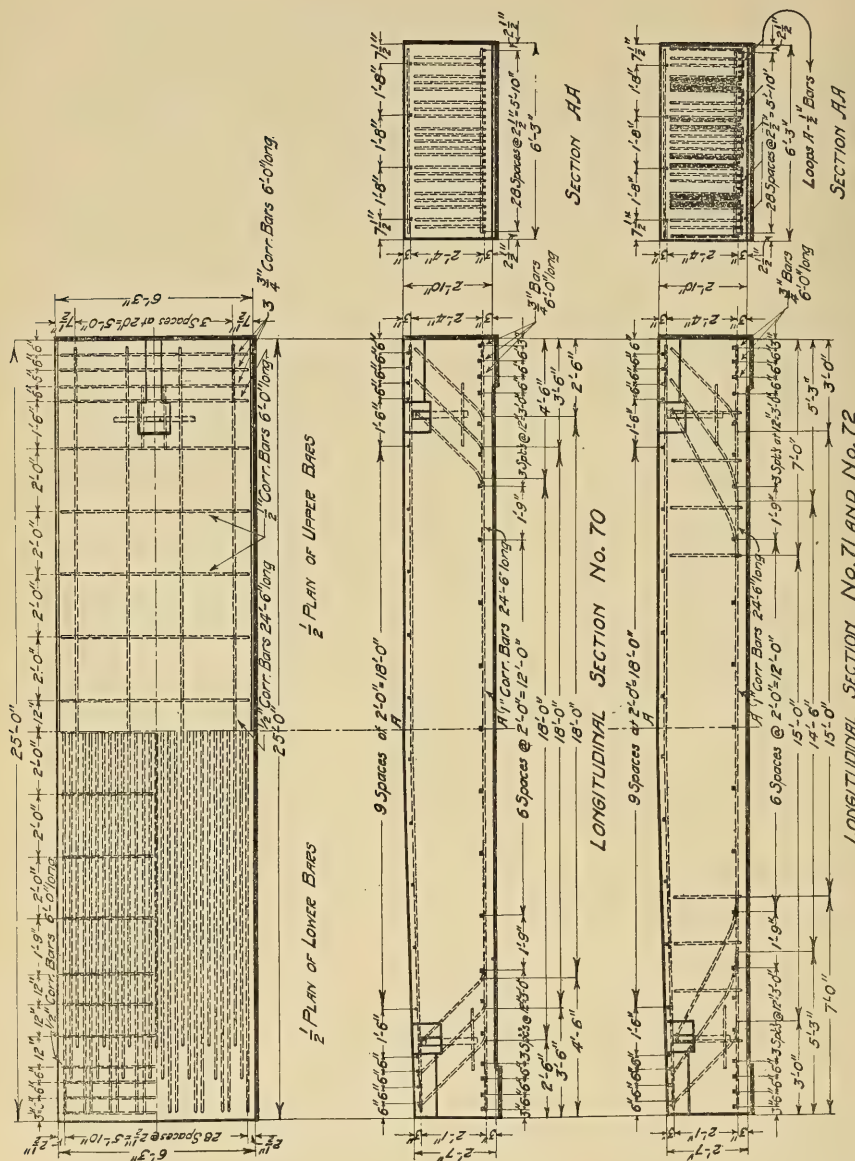


FIG. 2. PLAN OF FLOOR SLABS.

The longitudinal reinforcement was placed with the lower face of the bar 3 in. from the bottom surface of the slab. This makes the distance from the center of the reinforcing bars to the top of the slab at the middle $30\frac{1}{2}$ in. Although the depth to steel is less than this for one-half of the beam, the maximum bending moment is at the middle, and $30\frac{1}{2}$ in. will be used in the calculations, both in calculating the resisting moment and in calculating the shearing stresses. This is admissible in the latter case because of the slope of the upper surface. Twenty-nine 1-in. bars were used in each slab, the spacing being $2\frac{1}{2}$ -in. centers and $2\frac{1}{2}$ in. from the lateral faces. Counting the depth to the center of the steel as $30\frac{1}{2}$ in., the reinforcement amounts to 1.27%.

Two methods of placing the reinforcement and providing against diagonal tension were used. In No. 70 no stirrups were used and the bars were bent up at an angle of about 45° rather close to the end, as shown in Fig. 2. The bend for one-fourth of them started at about 2 ft. from the end, for another one-fourth at 3 ft., for another one-fourth at 4 ft., while the remaining bars were straight throughout the length of the slab. No. 70 was the last slab made in this way. In the slabs made after No. 70 (including No. 71 and No. 72) stirrups were used and the bars were bent up farther back from the end and less abruptly, giving a better distribution of the reinforcement (see Fig. 2). In one-fourth of the bars the inclination started at a point 6 ft. from the end, in one-fourth at 4 ft. from the end, and in one-fourth at 2 ft. from the end, while as before the remainder of the bars were horizontal. In these slabs U-shaped stirrups, made of $\frac{1}{2}$ -in. cup bars, were placed vertically at distances of $2\frac{1}{2}$ ft., 4 ft., $5\frac{1}{2}$ ft. and 7 ft. from the ends of the slabs. Each stirrup passed under and included 5 reinforcing bars and reached nearly to the top of the slab. In addition to the main reinforcing bars, 4 $\frac{1}{2}$ -in. cup bars were placed longitudinally 3 in. from the top of the slab, and transversely across the slab just above these were 12 $\frac{1}{2}$ -in. and 6 1-in. bars, while transversely across the bottom face just above the longitudinal reinforcement were 21 $\frac{1}{2}$ -in. bars. As the top reinforcement is only one twenty-fifth of one per cent it is not considered in the calculations of neutral axis position or of compressive stress in the concrete.

At the middle of the width of the slab and 2 ft. 6 in. from either end, heavy steel stirrups were anchored in the concrete. These

were designed to receive the device by which the slabs were lifted and placed.

6. *Composition and Fabrication.*—The proportions of the concrete were 1 cement, 2 sand, and 5 broken stone. The concrete was mixed in a No. 2 Smith machine mixer. There was no measurement of the amount of water used, but the concrete was mixed so wet that little or no tamping was required, or even feasible. The forms were strong and substantial and were well braced. The main longitudinal reinforcement was carefully placed and held by wires and blocks until the concrete was in place. The stirrups were held likewise. It is probable that more mortar was used in space below the bars on account of the difficulty in working around the bars, but there was no attempt made to vary the mixture in the slabs. The concrete was well spaded and stirred, especially around the reinforcing bars.

7. *Storage and Age.*—The slabs were left where made, exposed to the weather, from the time of fabrication until tested. The sides of the forms were removed after 48 hours. No. 70 was made Oct. 7 and No. 71 and No. 72 Oct. 8, 1907. The test of No. 70 was made April 15, and of No. 72 April 28, 1908. No. 71 was used in both tests. The slabs were therefore more than six months old. On account of the temperatures prevailing during the winter, the setting of the concrete during much of the time must have been slow, and the conditions were probably less favorable for hardening than is the case in ordinary 90-day laboratory tests.

8. *Testing Apparatus.*—Two slabs were tested against each other, the load being applied by hydraulic jacks. Fig. 3 shows the arrangement for testing. The lower of the two slabs was turned upside down and the second slab was placed above it, with bearing timbers between them at the ends. Two yokes were placed at the one-third points of the span length and four hydraulic jacks acting on the yokes supplied the load which forced the slabs together. The diagram (Fig. 3) and the photographs (Fig. 4 and 5) will assist in understanding the method of making the test. On account of the weight and size of the slabs and the great pressures developed, the usual devices and appliances of the testing laboratory could not be used. To overcome the natural unevenness of the slab, timber bearing plates—selected with a view to uniform quality—were used, and the considerable compression developed in these timbers permitted a good adjustment of the

load and pressures across the width of the slabs. The lower slab was supported on 12x12-in. timbers placed transversely across the slab at about 4 ft. from the ends. At either end a 12x12-in. oak timber was placed across the slab, the distance from center to center, 23 ft. 6 in., being considered the span length in the test.

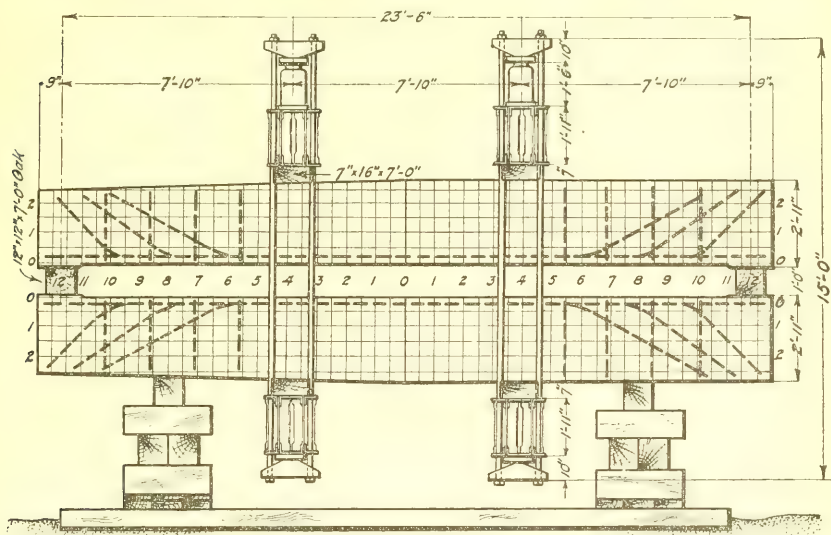


FIG. 3. ARRANGEMENT OF TEST APPARATUS.

The other slab was then placed right side up on top of these bearing timbers. The timbers were bedded in plaster of paris above and below in order to overcome small irregularities in the surface of the slabs. At the one-third points of the span length 7x16-in. yellow pine timbers were placed on top of the upper slab and also on the under side of the lower slab, and bedded in the same manner. Two 24-in. steel girders were placed at each one-third point on top of the timbers of the upper slab, and similar steel girders at the corresponding points below. These girders were discarded bridge beams made up of plates and angles and were stiff enough to give but slight deflection under the maximum load used. Sixteen wrought-iron rods (four rods at each of the four points) and thick cast-iron blocks were used to complete the connection. The hydraulic jacks were placed between the upper girders and the upper cast-iron blocks. The jacks used were of a nominal capacity of 100 tons each. The jacks had individual pumps and

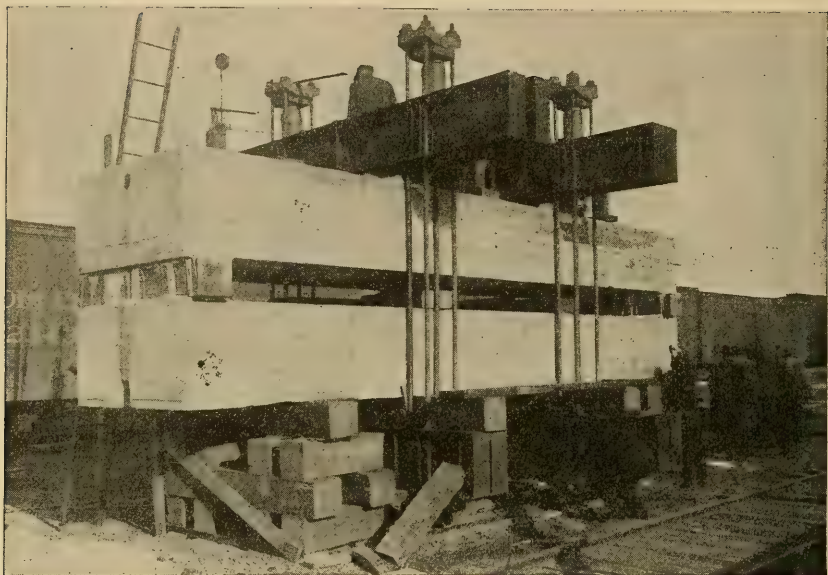


FIG. 4. VIEW SHOWING SLABS AND TESTING APPARATUS IN PLACE.



FIG. 5. GENERAL VIEW BEFORE TEST.

gauges. The jacks and gauges had previously been calibrated and the loads given in the tables include the necessary corrections. The load was applied in varying increments, but the amount was kept equally divided among the four jacks. The timber bearing plates gave a good distribution of the load over the full width of the slab and it seems sufficiently accurate to consider the distance from center to center of the supports as the span length of the beam.

It will be recognized that their great weight made the placing of the slabs a considerable undertaking. They were handled by means of a derrick car of 100 tons capacity and although this was the first experience of the men in handling the slabs, they did the work easily and satisfactorily. In lifting slab No. 71, however, the great weight on one rail caused one side of the temporary track to settle considerably, and, to prevent possible overturning of the car, the hoisting cable was slackened suddenly and the slab was dropped, falling four feet and striking on a pile of stone and against a rail. One corner of the slab was cracked, but so far as the tests show no other injury was done.

In the first test calipers were the only auxiliary measuring appliances used. In the second test the deflections of the beams were measured at mid-depth on either side by the usual thread-scale-mirror apparatus, giving readings accurate to within 0.01 in. An extensometer device for measuring longitudinal deformations similar to that used on test beams in the laboratory (see Bulletin No. 4 for description) was used in the test of No. 72. The gauged length was 50 in. The lower contact point was placed 2 in. above the bottom of the slab, the second contact point 28 in. above this, the upper extensometer 40 in. above, and the lower extensometer 10 in. below this lower contact point. This measuring apparatus was generally satisfactory, but for some reason there seems to have been a slip of the instruments during the test, as is discussed in a later article. A set of extensometers made up of Ames dials was placed on the lower slab, but these instruments are not suited to the purpose and did not prove satisfactory. A novel feature of the test was the use of extensometers to measure the deformations developed in a vertical direction. The position of these instruments is shown in the view given in Fig. 5, p. 10. The readings are useful in determining the load at which the vertical stirrups are brought into action.

9. *Method of Testing.*—The test required the services of a number of men. The four jacks were operated by eight men with three more to direct their work and record the loads. In the test

TABLE 1.

TEST DATA OF NO. 70 AND NO. 71.

Tested April 15, 1908.

The applied load includes the weight of the testing appliances but not the weight of the slab itself. The notes regarding cracks refer to No. 70.

Ref. No.	Applied Load pounds	Sum of Center Deflections of No. 70 and 71 inches		Remarks.
		E. Side	W. Side	
0	25 000	0.00	0.00	This load is used as the weight of testing apparatus and men plus the initial tension in the wrought-iron rods.
1	79 000	0.01	0.01	
2	113 000	0.05	0.03	
3	140 000	0.07	0.03	
4	174 000	0.12	0.08	
5	202 000	0.17	0.14	
6	230 000	0.19	0.16	
7	261 000	0.19	0.21	
8	299 000	0.24	0.28	
9	333 000	0.36	0.33	First tension crack noted in No. 70. Small tension crack at north load point.
10	364 000	0.37	0.40	
11	396 000	0.44	0.46	
12	432 000	0.47	0.53	Second tension crack near center of top slab.
13	468 000	0.57	0.60	First diagonal crack outside north load.
14	496 000	0.63	0.69	Numerous small tension cracks.
15	532 000	0.65	0.77	
16	561 000	0.78	0.83	
17	593 000	0.85	0.90	First diagonal crack lengthening rapidly and becoming more marked.
18	626 000	0.98	1.04	Cracking along the steel at south end.
19	661 000	1.06	1.08	
20	693 000	1.11	1.21	
				In the interval after 488 000 lb. other diagonal cracks appeared near the first one.
21	734 000	1.24	1.30	First diagonal crack opening wide.
22	768 000	1.34	1.45	
23	801 000	1.60	1.72	
				Failed suddenly along first diagonal crack, which reached top just inside the load point.

of No. 72 three observers and one recorder were stationed on either side of the beam and there were two men to examine it for cracks and to trace and mark their progress. The loads were

applied in varying increments, but the amount was kept nearly equally divided among the four jacks. The test was carried on without releasing the load. There was no difficulty in holding the load so as to maintain a constant reading on the gauges while measurements were being taken. In general, it may be said that the action of the jacks, the uniformity of the results, and other circumstances connected with the test indicate that the testing apparatus was trustworthy within the limits which may be considered necessary for the purpose of the test.

III. EXPERIMENTAL DATA AND DISCUSSION.

10. *Tables and Diagrams.*—Table 1 and Fig. 6 give the combined deflections of No. 70 and No. 71. The high wind prevailing at the time of the test made it impracticable to use the apparatus provided and the deflections measured were the sum of the center deflections of the upper and lower beams. It seems probable that the individual deflections of the slabs were nearly the same except near the end of the test. Table 2 gives data of No. 72 and Table 3 gives values calculated from these data. The column headed "Stress in Steel from Deformation" is obtained by multiplying the deformation of the steel determined from the extensometer reading by 30 000 000. The discussion of the trustworthiness of these readings given in a later paragraph indicates that these values are somewhat higher than they should be. The column headed "From Bending Moment" is calculated by equating the bending moment due to the applied load and the approximate value for the resisting moment, $0.87Afd$, where A is the area of the reinforcing bars in square inches, d is the distance from the top of the slab to the center of the steel, here called $30\frac{1}{2}$ in., and f is the stress in the steel in pounds per square inch. This calculation does not include the weight of the beam. In comparing the values in the last two columns it should be remembered that the extensometer readings really started from a load of 25 000 lb., since a load equal to this was upon the slabs at the initial reading of the instruments. The reference numbers in Tables 2 and 3 refer to those marked on the beams during the progress of the test. These numbers are shown on some of the

photographs. In Fig. 7 the deformations and deflections of No. 72 are plotted. Fig. 10 gives the vertical deformations for No. 72 and Fig. 11 for No. 71. Table 5 gives miscellaneous calculated values.

11. *Phenomena of the Tests.*—As is usual in such tests, minute vertical cracks became visible in the concrete on the tension

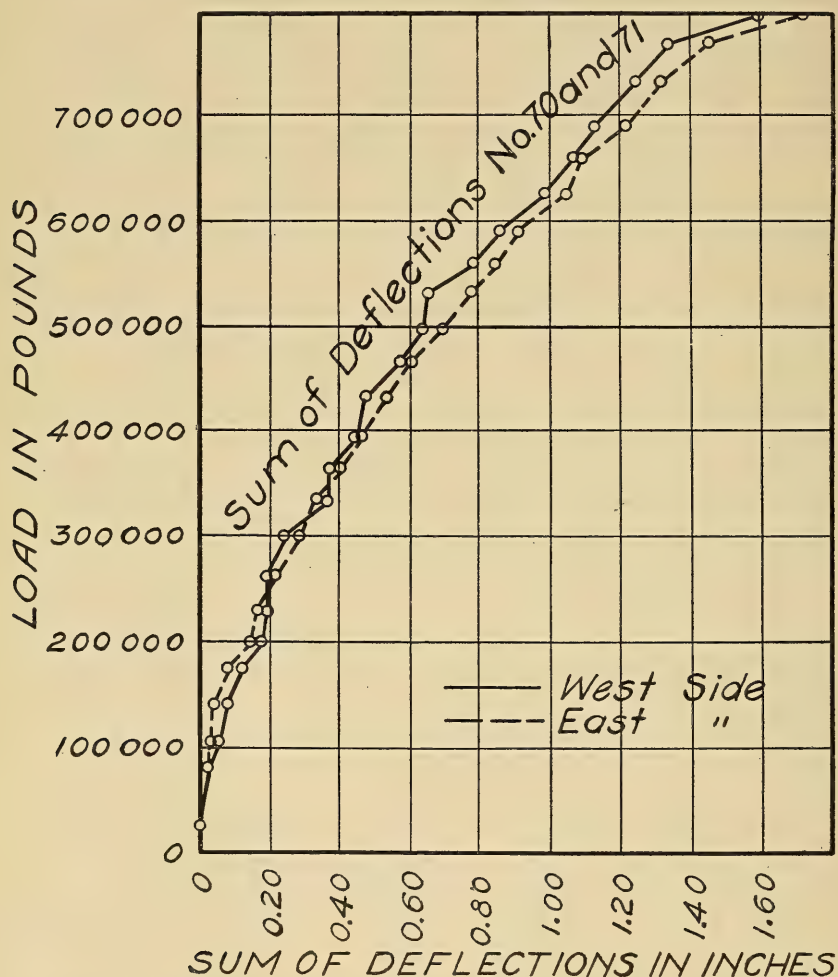


FIG. 6. SUM OF CENTER DEFLECTIONS OF NO. 70 AND NO. 71.

TABLE 2.

TEST DATA OF NO. 72.

Tested April 28, 1908.

The applied load includes the weight of the testing appliances but not the weight of the slab itself.

Ref. No.	Applied Load pounds	Average Deflection inches	Average Extensometer Readings inches		Vertical Extensometer Readings inches		Remarks
			Top	Bottom	South	North	
0	25 000*	0.00	.000	.000	.0000	.0000	The "Average Extensometer Readings" are for a gauge distance of 50 in.
1	45 000	0.00	.000	.000	.0000	.0000	
2	67 000	0.01	.001	.001	-.0001	.0000	
3	103 000	0.02	.004	.004	-.0001	-.0003	
4	131 000	0.03	.006	.006	-.0001	-.0002	
5	159 000	0.05	.008	.010	-.0002	-.0003	Short tension crack just inside south load point.
6	189 000	0.10	.011	.016	-.0002	-.0002	Numerous tension cracks.
7	220 000	0.11	.015	.023	-.0002	-.0002	Vertical cracks just outside each load point.
8	253 000	0.13	.017	.030	-.0002	+.0002	More cracks outside of load points.
9	285 000	0.17	.019	.037	-.0003	+.0004	
10	317 000	0.18	.021	.044	-.0004	+.0003	
11	350 000	0.24	.024	.050	-.0005	+.0002	
12	382 000	0.28	.028	.057	-.0006	.0000	
13	414 000	0.31	.032	.064	-.0004	+.0003	Marked diagonal cracks.
14	447 000	0.36	.037	.070	-.0001	+.0011	
15	500 000	0.40	.043	.075	.0000	+.0014	
16	514 000	0.43	.046	.081	+.0007	+.0021	
17	578 000	0.52	.053	.094	+.0023	+.0045	
18	643 000	0.61	.062	.107	+.0047	+.0079	Tension cracks opening wide.
19	704 000	0.70	.071	.122	+.0083	+.0119	
20	780 000	0.88	.084	.156	+.0138	+.0152	
21	811 000	1.32	.132	.270	+.0170	+.0181	
22	840 000	1.60					Maximum load. Tension failure.

*This load of 25 000 lb. is assumed as the weight of testing apparatus and men and the initial tension of rods. After jacks were in operation all but 9000 lb. was indicated by readings of the gauges.

side of the beam between the load points as the load was applied and on the outside of the load points at higher loads. These cracks grew in length and became more marked as the load was increased. In the first test (No. 70 and 71 together), in which the beams did not fail by the reinforcing steel stretching up to the yield point, these cracks closed upon the release of the load to such an extent as to be scarcely detectable. In No. 70 (not white-washed) the first tension crack was noted at an applied load of 333 000 lb. and numerous tension cracks at 496 000 lb. In No. 72 (whitewashed surfaces) the first tension crack was noted at 159 000 lb. and numerous tension cracks at 189 000 lb. It will be seen

TABLE 3.

RESULTS OF TESTS OF No. 72.

The applied load includes the weight of the testing appliances but not the weight of the slab itself.

Ref. No.	Applied Load pounds	Average Deflection inches	k of Neutral Axis	Unit Deformation		Stress in Steel* lb. per sq. in.	
				At Steel	At Top Fiber	From Deformation	From Bending Moment
0	25 000	0.00	.000	.00000	.00000		1 500
1	45 000						2 700
2	67 000	0.01	.549	.00001	.00001	300	4 100
3	103 000	0.02	.549	.00004	.00005	1 200	6 300
4	131 000	0.03	.549	.00007	.00008	2 100	8 000
5	159 000	0.05	.459	.00012	.00010	3 600	9 700
6	189 000	0.10	.392	.00020	.00013	6 000	11 500
7	220 000	0.11	.379	.00025	.00018	7 500	13 400
8	253 000	0.13	.326	.00035	.00019	10 500	15 300
9	285 000	0.17	.289	.00049	.00020	14 700	17 400
10	317 000	0.18	.263	.00059	.00021	17 700	19 300
11	350 000	0.24	.265	.00067	.00024	20 100	21 400
12	382 000	0.28	.273	.00077	.00029	23 100	23 300
13	414 000	0.31	.280	.00086	.00033	25 800	25 300
14	447 000	0.36	.300	.00093	.00040	27 900	27 300
15	480 000	0.40	.330	.00098	.00048	29 400	29 300
16	514 000	0.43	.326	.00106	.00051	31 800	31 400
17	578 000	0.52	.324	.00123	.00059	36 900	35 300
18	643 000	0.61	.334	.00140	.00070	42 000	39 300
19	704 000	0.70	.335	.00159	.00080	47 700	42 900
20	780 000	0.88	.306	.00204	.00096	61 200	47 600
21	811 000	1.32	.271	.00360	.00132	108 000	49 600
22	840 000	1.60					

*Add 2500 lb. per sq. in. for stress due to weight of slab.

from the photographs (see views in frontispiece) that some of these minute cracks reached points 12 inches from the top of the slab before the reinforcement was stressed to its yield point. Even when they had thus grown, their width at the bottom was minute.

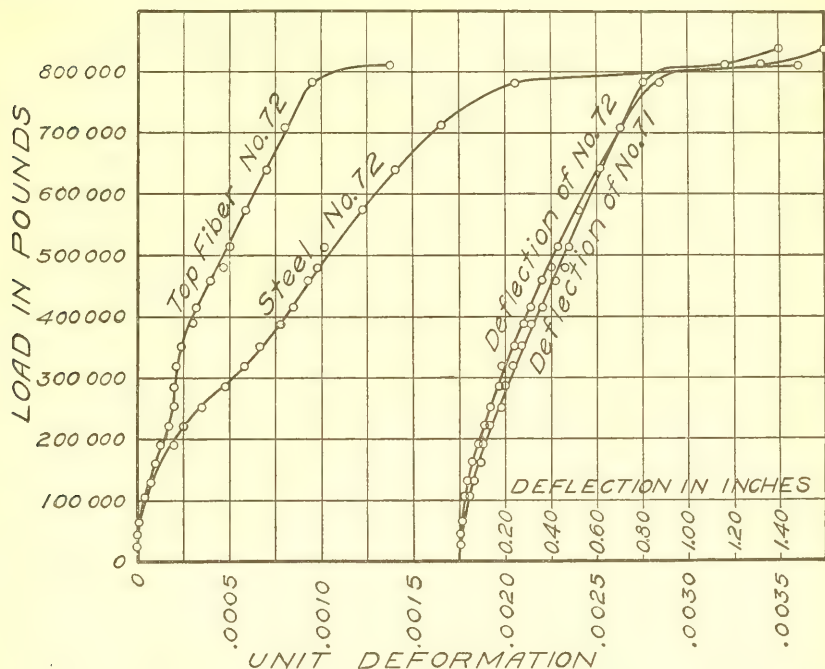


FIG. 7. LOAD-DEFORMATION AND DEFLECTION DIAGRAM FOR NO. 72.

Outside the load points (i. e., in the outer thirds of the beam length) diagonal cracks appeared, frequently forming from the top of the vertical crack already visible and extending diagonally upward and downward at the same time. In No. 70 the first diagonal crack was visible at an applied load of 468 000 lb., and in No. 72 the cracks took a distinctly diagonal direction at 447 000 lb. The diagonal cracks developed with the addition of load, but the form of this development was not the same in No. 70 as in No. 72. In No. 70, the slab without stirrups and having the bars bent up abruptly near the ends, one main diagonal crack formed outside each load point making an angle of about 35° with the horizontal. While a few others became visible, these main cracks enlarged

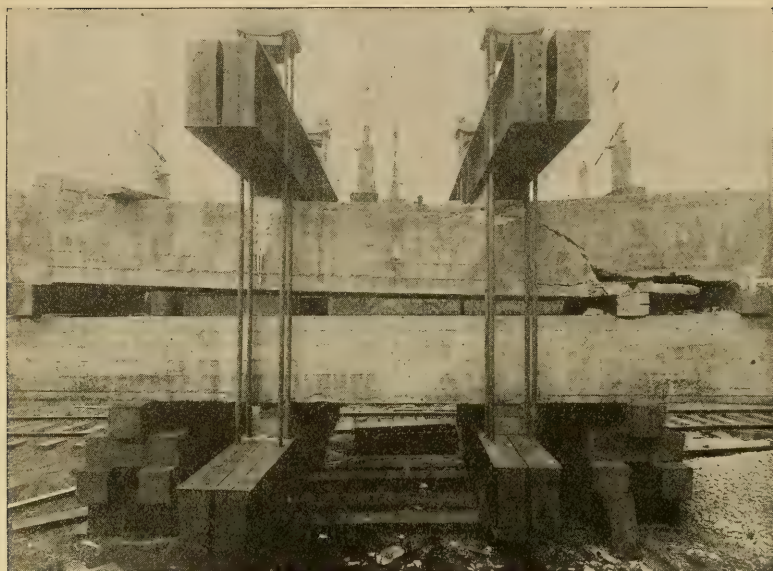


FIG. 8. MANNER OF FAILURE OF NO. 70.

and the final failure of the slab was along one of them. In No. 72 (whitewashed surfaces) marked diagonal cracks appeared at 447 000 lb. and became numerous and extended at 480 000 lb. Some of these finally extended to within 12 inches of the top face of the slab, but they remained small and fine and were well distributed over the beam. They lacked the growth and concentration which are apparent in failures by diagonal tension when reinforcing bars are carried straight to the end of the beam or nearly to the end and when stirrups are not used. The final failure of these beams is discussed in the next paragraph.

12. *Manner of Failure.*—The manner of failure was quite different in the two beams, No. 70 and No. 72. In the case of No. 70, although it is evident from calculations that the steel of the reinforcing bars had nearly reached the yield point, there was no evidence of this in the appearance and action of the beam at the time of failure. At a load of 593 000 lb. the main diagonal crack outside of one load point was seen to be extending rapidly and at 734 000 lb. it was opening wide. It was evident for some time that the beam was on the verge of total failure. Finally it failed suddenly by diagonal tension, so called shear failure, at an applied

load of 801 000 lb., the diagonal crack having suddenly extended to the top of the slab and opened wide. At the same time the horizontal crack which had formed along the reinforcing bar (by the action of vertical tension) lengthened and the bar pulled away from the concrete above. The characteristics of this test were the formation of main diagonal cracks at either end and the final sudden failure of the beam by diagonal tension. Fig. 8 shows the form of failure of No. 70, and Fig. 9 gives a nearer view of the main diagonal crack from the other side of the beam after the load was released.

In No. 72 the presence of the stirrups and the changed position of the reinforcing bars at the ends were sufficient to resist the diagonal tension developed. Although the diagonal cracks were numerous and although some of them finally extended to within 12 inches of the top face of the slab, (see views in frontispiece) they remained small and fine and were well distributed. They lacked the growth and concentration which are apparent in failures by diagonal tension. The tension

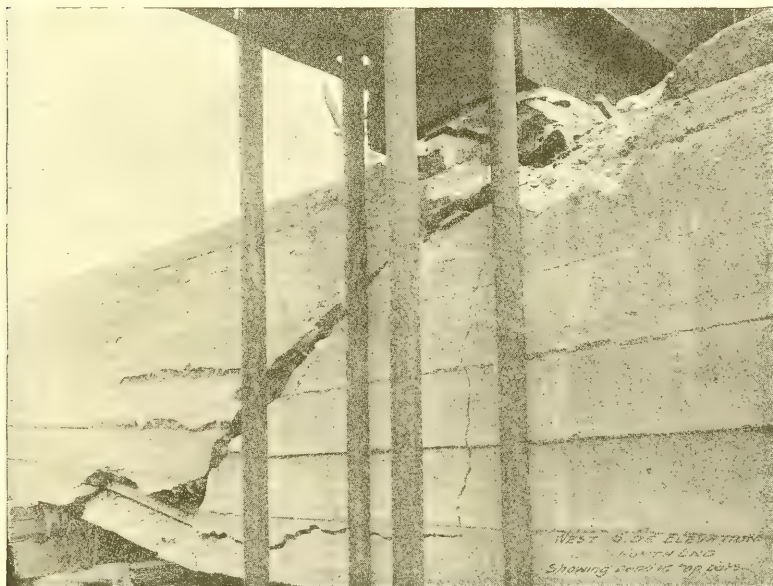


FIG. 9. MAIN DIAGONAL CRACK AT NORTH END OF NO. 70 AFTER RELEASE OF THE LOAD.

TABLE 4.
MISCELLANEOUS CALCULATED STRESSES.

Ref. No.	Applied Loads pounds	Calculated Tension in Steel lb. per sq. in.	Calculated Shearing Stress lb. per sq. in.	Remarks.
No. 70.				
9	333 000	22 800		First tension crack noted.
13	468 000		125	First diagonal crack noted.
17	593 000		156	One main diagonal crack extending rapidly.
21	734 000		195	One main diagonal crack opening wide.
23	801 000	51 700	209	Failure by diagonal tension.
No. 72.				
5	159 000	12 500		First tension crack noted.
14	447 000		120	Marked diagonal cracks noted. Marked increase in vertical extensometer readings.
15	480 000		128	Diagonal cracks numerous and extending.
16	514 000		138	Marked increase in second extensometer readings.
21	811 000	52 400		Tension cracks opening wide.
22	840 000	54 000	220	Failure by tension in steel.

The calculated stress in steel and the calculated vertical shearing stress include the stress due to weight of slab.

cracks in the middle third of the beam extended upward as the load was increased and at 811 000 lb. nine of these were visible to within 12 inches of the top of the slab. At this load the tension cracks were opening wide, indicating that the yield point of the reinforcing steel had been passed. Above 800 000 lb. the deflection increased rapidly. The maximum load applied was 840 000 lb., the indicated load then falling off when the deflection was increased. It seems evident that a load of 811 000 lb. would not have held long, and the maximum load applied held but momentarily. The characteristic of the test of No. 72 was slow failure by tension of the steel, without sign of compression failure and without sign of impending failure by diagonal tension. The effectiveness of the stirrups and the bending of the bars at

ends are evident in this test. This is the more important as the diagonal tension failure may give little warning and may develop with repetitions at lower loads. It may be noted that on the next day the test was continued until there was a deflection in No. 72 of $3\frac{1}{2}$ in., the load applied ranging somewhat under 800 000 lb. Upon release of load there was a recovery of 1 in. in the deflection. At this time the stretch of the steel and the consequent concentration of the compressive stress were so much that there was considerable crushing of the concrete. Slab No. 71, which was used as the lower beam in both tests, seems not to have suffered from the first test, the cracks closing up upon release of load. In the second test the tension cracks opened up at the maximum load and there was evidence of the steel being stretched beyond the yield point. It should be noted that on account of its inverted position and the location of the supports the bending moment developed in the lower beam was somewhat smaller than that in the upper beam. There was no sign of impending failure by diagonal tension.

13. *Position of Neutral Axis.*—The proportionate depth of the neutral axis calculated from the observed deformations on the usual assumptions is given in Table 3. The position of the neutral axis obtained from these measurements seems abnormally high, averaging during the later stages of the test about 0.34 of the distance from the top of the slab to the center of the reinforcing bars. The proportionate depth of the neutral axis for the amount of reinforcement used, based upon ordinary assumptions, would be 0.43 for a ratio of 12 between the moduli of elasticity of steel and of concrete, and 0.47 for a ratio of 15 (see Bulletin No. 4, p. 16). Even for a ratio of 10 the proportionate depth would be about 0.40. An examination of the original data between the loads of 220 000 lb. and 320 000 lb. makes it apparent that there was a slip of the instruments on the compression side of the beam at different times between these loads. The conditions under which the extensometers were used were not favorable for accurate results. Judging from the data and from experience in other tests, it is probable that between loads of 400 000 and 750 000 lb. the position of the neutral axis averaged about 0.40 of the depth. Even this value is suggestive of dense, stiff concrete. It should be noted that with such a slip of the upper extensometers the real compressive deformation would be

greater than that given in the diagram and also that the real tensile deformation would be somewhat less.

14. *Tensile Stresses in Reinforcement.*—The tension in the main reinforcing bars at the middle of the span length in No. 70, calculated by the usual methods (using the formula $0.87 A_f d$ before referred to) and including the effect of the weight of the beam, was 51 700 lb. per sq. in. at the maximum applied load of 801 000 lb. As the tension cracks in the concrete closed up after the failure of the beam and the release of the load, it would seem that the steel had not been stressed beyond its yield point. In No. 72 the calculated tension in the reinforcing bars, including the effect of the weight of the slab, was 54 000 lb. per sq. in. for the maximum applied load of 840 000 lb. For the applied load of 811 000 lb., where the yield point had evidently been passed, the calculated tension is 52 300 lb. per sq. in. It seems probable that the yield point was reached at the applied load of 780 000 lb. These values check up with the yield point of the material within the limits of variation of such materials.

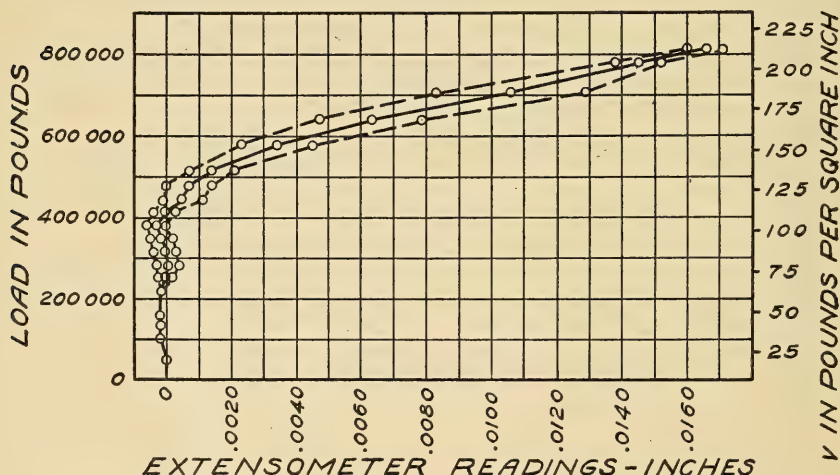


FIG. 10. VERTICAL DEFORMATION DIAGRAM FOR NO. 72.

15. *Vertical Shearing Stresses and Resistance to Diagonal Tension.*—As the diagonal tensile stress which is developed in the concrete depends upon the horizontal tensile stress existing in the concrete and the effect of stirrups, its amount is indefinite

and the value of the vertical shearing stress may be used as a means of comparison for the resistance to diagonal tension. The vertical shearing stress has been calculated by the formula

$$v = \frac{V}{bd'}$$

where V is the total vertical shear at the point where the crack begins, including the shear due to weight of the slab, b is the width of the slab, and d' is the distance from the center of the reinforcing bars to the center of gravity of the compressive area of the concrete, called here 0.87 of the depth to center of reinforcing bars. Calculated values for the various loads are given in Table 4.

In No. 70 the value of the vertical shearing stress at the breaking load is 209 lb. per sq. in. This is a high value for a beam without stirrups and having bars bent up abruptly at the end and shows a good quality of concrete. The value of v for the applied load of 468 000 lb., where the first diagonal crack was noted, is 125 lb. per sq. in.; and for the applied load of 593 000 lb., where the main diagonal crack was seen to lengthen rapidly and become more marked, it is 156 lb. per sq. in.

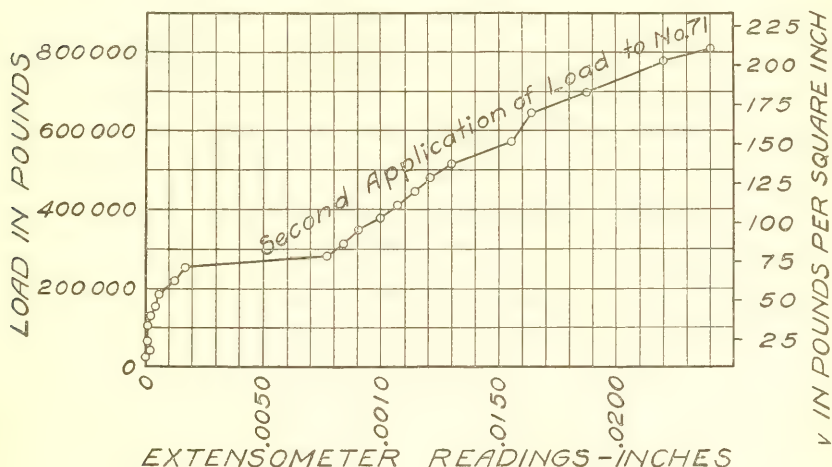


FIG. 11. VERTICAL DEFORMATION DIAGRAM FOR NO. 71.

In No. 72 the value of the vertical shearing stress v at the maximum load applied is 220 lb. per sq. in. As there was no sign of impending failure and as the diagonal crack closed up after the failure of the beam it is seen that the provision against failure by diagonal tension was very satisfactory. The readings of the vertical extensometers (given in Table 2 and platted in Fig. 10) are of interest. The position of these extensometers is shown in Fig. 5. The gauged length is about 28 in. By reference to Fig. 10, it will be seen that for the lower loads the instruments show a vertical shortening and that the deformation soon changes to elongation, this change taking place at loads corresponding to loads which gave diagonal cracks in the test of No. 70. The amount of this elongation rapidly increases with the application of further load. The marked increase in the reading of the extensometer on the south half of the beam begins at the load of 414 000 lb. and is accompanied with an evident development of diagonal cracks. A similar development was noted in the north half of the beam at a load of 480 000 lb. The arrangement of the instruments and our information of the position of the stirrups were not such as will permit the readings to be used as the basis of calculations, but they serve as an indication of what is going on in the interior of the beam. It is of interest also to note in the diagram of extensometer readings taken on No. 71 in its second test (shown in Fig. 11) that on this second application of the load the line of vertical deformation beyond a load of 250 000 lb. would, if extended, pass through the origin. This is the position it would have if the stirrups take vertical tension proportional to the load from the beginning.

16. *Compressive Stress in the Concrete.*—The compression in the upper fiber, calculated by the ordinary methods, runs up to a high figure, as is usual in tests of beams having a considerable amount of reinforcement and adequate provision for web stresses. Assuming the neutral axis to be 0.43 of the depth to the reinforcing bars and using the ordinary straight-line formula, the compressive stress in No. 72 at the maximum load was 3190 lb. per sq. in. Using the parabolic formula and considering that the compressive deformation is one-half of the ultimate compressive deformation of the concrete, the calculated stress is 2870 lb. per sq. in. Even at this high calculated stress there was no sign of compression failure and none was apparent until after the steel

had stretched beyond its yield point. Comparing the deformation developed with that of other tests it seems probable that the beam would have taken at least one-third more compressive stress before failure in compression at first load would have resulted. Of course, repeated applications of any such high load might soon have injured the beam. A further evidence of the strength of the concrete is seen in the later test referred to, where after the maximum load had been reached and the steel was stretched well beyond its yield point the beam held a load of nearly 800 000 lb. until the deflection reached $3\frac{1}{2}$ in before any considerable amount of crushing of the concrete took place.

17. *Remarks.*—The tests showed that the concrete was of excellent quality and that the slabs acted similarly to high-grade test beams made and tested in laboratories. The uniformity and regularity of these large beams were shown in various ways in the tests. As these particular slabs may be presumed to be fairly representative of the slabs fabricated for the work, the tests ought to add confidence in the quality and soundness of the reinforced concrete used in this work and in other work of large magnitude made under as careful conditions. The action of the concrete in compression under the high stresses developed was quite satisfactory. The tests show the effectiveness of stirrups and of the methods used in bending up bars at the ends in resisting diagonal tension. Diagonal tension weakness is particularly undesirable because of the possibility of sudden failure and of injury after repeated applications of the load and because of the difficulty of detecting incipient failure when the sides of the beam are not available for inspection. Ample safety against these conditions is important and means should be provided to resist the diagonal tension. Failures by tension of the steel and by compression of the concrete give warning through abnormal deflections and in other ways and are less likely to lead to serious results. Also, it seems evident from these tests that the action and properties of reinforced concrete beams made under efficient supervision do not differ much from those of laboratory test beams.

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BULLETIN NO. 29

TESTS OF REINFORCED CONCRETE
BEAMS: RESISTANCE TO
WEB STRESSES

SERIES OF 1907 AND 1908

BY

ARTHUR N. TALBOT



UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

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UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

BULLETIN No. 29

JANUARY 1909

TESTS OF REINFORCED CONCRETE BEAMS:
RESISTANCE TO WEB STRESSES.
SERIES OF 1907 AND 1908.

BY ARTHUR N. TALBOT, PROFESSOR OF MUNICIPAL AND SANITARY
ENGINEERING AND IN CHARGE OF THEORETICAL
AND APPLIED MECHANICS.

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C. BEAMS WITH STIRRUPS

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I. INTRODUCTION.

1. *Scope of Bulletin.*—Tests of reinforced concrete beams made in various laboratories have provided a considerable fund of information bearing on the part played by the tensile resistance of the longitudinal reinforcement and the compressive resistance of the concrete. The data on resistance to web stresses are less definite and less complete. In beams having a depth relatively large in comparison with their length, the resistance to web stresses becomes very important, and diagonal tension and bond stresses may control the strength of the beam. It was with a view of obtaining further information on web stresses and on the part taken by concrete, bent reinforcement, and stirrups in resisting web stresses that these series of tests were undertaken.

The tests were carried on during the years 1906-7 and 1907-8. Tests were made on (1) beams without web reinforcement, (2) beams with reinforcing bars bent up in various ways along the outer thirds of their length, and (3) beams with vertical U-shaped stirrups. Four span lengths were used. Various amounts of longitudinal reinforcement from 1 % to 2.2 % were used. Tests were made with concrete at different ages with a view of determining the effect of early removal of forms. Six mixtures of concrete were used, ranging from very lean to very rich. Compression tests of cubes and cross-bending tests of small concrete beams, called here control beams, were made with a view of determining whether a relation between the strength of the reinforced concrete beams and the control pieces might be found which would be useful as a control test in building construction. 117 beams were made in 1907 and 71 beams in 1908. 106 of these were without web reinforcement, 27 had the reinforcing bars bent up in one form or another, and 55 had stirrups for the web reinforcement. It was found impossible to carry out the tests exactly as planned, and the data of the tests do not furnish a basis for as definite conclusions as had been hoped. However, the tests throw light on the phenomena of web resistance and the results have a bearing on the principles of design. Reference may also be made to the values of web resistance reported in Bulletins No. 4 and No. 14.

2. *Acknowledgment.*—The investigations were made in the Laboratory of Applied Mechanics of the University of Illinois as a part of the work of the University of Illinois Engineering Experiment Station. Much of the work of making the tests of the beams was done as thesis work by senior students in civil engineering. The following list gives the names of the students engaged in this work. They are entitled to credit for the care, skill, and interest which they showed in the work, as well as for the suggestive way in which they discussed the results.

Tests of Reinforced Concrete Beams.

Series of 1907.

Chester A. Foreman, Carl James, Leon B. Kinsey. Resistance to web stresses.

Frank Howard Reynolds. Effect of storage and consistency.

Francis Cyrus Bagby, Augustus Bacon Casey. Modulus of elasticity for various mixtures.

Edwin Walker Buxton. Effect of bending and anchoring reinforcing bars.

William Hadden Richardson. Flexural strength of concrete.

Series of 1908.

Raymond Clark Pierce, Hiram James Powers, Robert Milton Van Petten. Resistance to web stresses.

Arthur Linn Bear, Samuel Charles Campbell. Modulus of elasticity for various ages and mixtures.

Otto Arthur Dicke. Tests of flexural strength of concrete.

The immediate supervision of the work of making the beams and of the work of conducting the tests was given by D. A. Abrams, Associate in the Engineering Experiment Station. Acknowledgment is also made to H. E. Murdock, Instructor in Theoretical and Applied Mechanics, for assistance in the preparation of this bulletin.

3. *Classification of Stresses.*—A brief statement of the methods of analysis and of the formulas used in the calculations and discussions given in this bulletin, so far as is necessary to make clear the matter presented, is given in the following pages. This presentation necessarily must be incomplete, and some of the statements made are given tentatively and are to be taken with limitations. In the discussion freedom from end restraint of any kind is always assumed, and in the tests care was exercised to secure the same condition.

In the analysis of beam action, two classes of stresses are considered; (1) the horizontal or longitudinal component stresses (tension and compression) which are used in the calculation of the resisting moment of the section, and (2) diagonal, vertical, and horizontal stresses, like shear and diagonal tension and diagonal compression, which constitute what are frequently termed web stresses. The first named stresses and the resisting moment resulting therefrom in many cases measure the ability of the beam to carry the required load. With reinforced concrete beams, however, the second type of stresses under conditions which are not unusual, may be the limiting element in the strength of the beam, and the resistance to such stresses therefore constitutes an important feature of a well designed beam. These web stresses may include bond stress (resistance to slip of reinforcing bar in the concrete), shearing stresses in various directions, and tensile stresses and compressive stresses in directions other than that parallel to the axis of the beam. The calculation of the amount and distribution of the web stresses is complicated, and for use in design it will be convenient to use certain assumptions and approximations. The discussion will be made under three heads: (1) Beams without Web Reinforcement. (2) Beams with Bars Bent Up, and (3) Beams with Stirrups.

4. *Notation.*—The following notation will be used. Reference may also be made to Fig. 1.

b = breadth of rectangular beam.

d = distance from the compression face to the center of the metal reinforcement.

A = area of cross section of longitudinal reinforcement.

$p = \frac{A}{bd} =$ ratio of area of metal reinforcement to area of concrete above center of reinforcement.

o = circumference or periphery of one reinforcing bar.

m = number of reinforcing bars.

E_s = modulus of elasticity of steel.

E_c = initial modulus of elasticity of concrete in compression.

$n = \frac{E_s}{E_c} =$ ratio of two moduli.

f = tensile stress per unit of area in metal reinforcement.

c = compressive stress per unit of area in most remote fiber of concrete.

d' = distance from the center of the reinforcement to center of gravity of compressive stresses.

j = ratio d' to d . $d' = jd$.

ΣX = summation of horizontal compressive stresses.

M = resisting moment at the given section.

s = horizontal tensile stress per unit of area in the concrete.

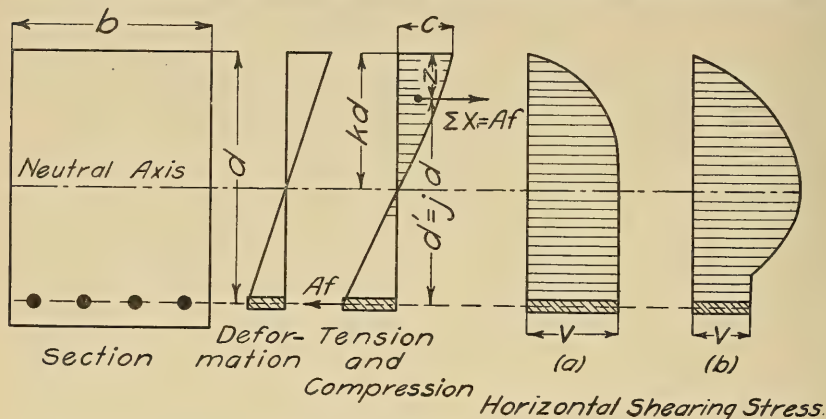


FIG. 1. DISTRIBUTION OF STRESSES OVER CROSS-SECTION OF BEAM.

t = diagonal tensile stress per unit of area in the concrete.

u = bond stress per unit of area on the surface of the reinforcing bars.

v = vertical shearing stress and horizontal shearing stress per unit of area in the concrete.

5. *Resisting Moment.*—In the calculation of the resisting moment of reinforced concrete beams, it is the usual practice to neglect the tension in the concrete. At sections where the bending moment is a maximum this is allowable. With this assumption the expression for the resisting moment (see Fig. 1) is

$$M = Afd' = Afjd \dots \dots \dots (13)$$

jd is here used as the equal of d' . Its exact value is to be determined by the methods given on pages 16 and 17 of Bulletin No. 4. The value of j for the beams discussed in this bulletin varies from 0.80 to 0.88.

For sections nearer the point of zero bending moment (as for example, in simple beams near the supports), tension exists in the concrete from the neutral axis downward for some distance and even to the bottom of the beam. It is evident that for such

portions of the beam the presence of this tension should be taken into consideration in the discussion of web resistance. It is also evident that the formula for resisting moment given above, if applied to such portions of the beam, would involve considerable error.

6. *Beams without Web Reinforcement: Bond Stresses.*—In order to have beam action there must be a proper web connection between the tension and the compression portions of the beam. When there is no metallic web reinforcement, the concrete acts alone as this web. It is clear, of course, that the amount of stress in the reinforcing bars and also in the compression area of the concrete varies from cross-section to cross-section along the length of the beam. The increment of tension and that of compression gained between consecutive sections must be connected by means of this web; that is, the increment in the tensile stress in the reinforcing bars must be transferred to or connected with the increment of the compressive stress in the concrete.

In transmitting the increment of stress from the reinforcing rods to the surrounding concrete there is developed a tendency of the rods to slip in the concrete, and the amount of resistance to slip thus developed is called bond and will be measured in terms of the area of surface in contact with the concrete. It will be seen that the total bond developed on the surface of the bars in one inch of length is equal to the total change in the amount of tension in the bars for the same inch of length. Bond may be compared to the action of the rivets joining flange to web in a riveted steel girder, except that in a reinforced concrete beam the contact is continuous.

For beams in which the reinforcement is horizontal or straight throughout, the formula for bond may be derived as follows if the tensile strength of the concrete in resisting longitudinal stresses is neglected. At any vertical section of the beam the value of the resisting moment is given by equation (13) ($M = Afd'$), where f is the unit stress in the steel at the section considered and d' is the distance from the center of the steel to the center of compression in the concrete. Differentiating this equation, $\frac{dM}{dx} =$

$A \frac{df}{dx} d'$. By the principles of mechanics of beams, $\frac{dM}{dx} = V$, where

V is the total vertical shear at the given section (reaction at support minus loads between the support and the section considered). Substituting and transposing,

$$\frac{Adf}{dx} = \frac{V}{d} \dots \dots \dots (16)$$

Now the derivative $\frac{Adf}{dx}$ expresses the rate of change of the total tensile stress in the reinforcing bars at the section under consideration; it is given in terms of a unit of length of beam (pounds per inch of length) and measures what is transmitted to the concrete by the bond. Using m as the number of bars, o as the efficient circumference or periphery of one bar, the total surface of bars for one inch of length of beam is mo and the bond stress developed is mu , where u represents the bond developed per unit of area of surface of bar. Equating this to the value of the derivative in equation (16) and solving,

$$u = \frac{V}{mod} \dots \dots \dots (17)$$

Attention should be called to the fact that equation (17) is derived for beams having the reinforcing bars straight throughout their length, that tension in the concrete is neglected, and that it must be modified in case any or all of the bars are bent up. If the longitudinal tension in the concrete is considered, as it may need to be near the ends of the beam it will be found that the bond developed will be less than given by the formula if the value of f is derived directly from the bending moment. In one portion of the beam there will be tension in concrete from the neutral axis down to the bottom of the beam; close to this, tension will exist for but a short distance below the neutral axis. For the portion of the beam between these two conditions the bond developed will be even more than that given by the formula.

7. *Vertical and Horizontal Shearing Stresses in Beams without Web Reinforcement.*—It is shown in the mechanics of beams that there exists throughout a beam vertical and horizontal shearing stresses which vary in intensity, and that at any point in a beam the vertical shearing unit-stress is equal to the horizontal shearing unit-stress there developed. As noted in the discussion of bond, the total tension in the reinforcing bars varies along the length of the beam, as does also the total compressive stress. The

horizontal shearing stress is seen to be necessary in order that the increments or increase of the total tensile stresses in the reinforcing bars (transmitted to the surrounding concrete by the bond stresses), may be transmitted to the corresponding increments of compression in the compression area of the concrete. The concrete thus forms the stiffening web of the beam.

The amount of this horizontal tensile stress so transmitted from the reinforcing bars per unit of length of beam is by equation (17) $mou = \frac{V}{d'}$. Consider this distributed over a horizontal section just above the plane of the bars for a unit of length of beam, and call the horizontal shearing unit-stress v . The shearing resistance per unit of length of beam thus developed is then bv , and equating this to mou ,

$$v = \frac{V}{bd'} \dots\dots\dots (18)$$

This equation gives the horizontal shearing unit-stress, and therefore also the vertical shearing unit-stress, at a point just above the level of the reinforcing bars. As no tension is here considered as acting in the concrete, there will be no change in the intensity of the horizontal and vertical shearing stresses between this level and the neutral axis. Above the neutral axis the intensity of the shearing stresses will decrease by the law of change of horizontal shearing stresses for homogeneous rectangular beams modified to suit any variation from the straight-line stress-deformation relation. The distribution of the intensity of the horizontal shearing stress over a vertical section is represented in Fig. 1 (a).

As d' may range from $0.80d$ to $0.88d$ for the conditions of the beams here considered, the shearing stress by equation (18) is from 25% to 13% more than the average shearing stress or than that found by considering the stress to be uniformly distributed over a vertical section extending down to the center of the reinforcing rods.

The presence of longitudinal tension in the concrete modifies the distribution of the vertical shear, increasing the shearing stress at the neutral axis and decreasing it at the level of the reinforcement. If the total tension taken by the concrete be estimated, and also the position of the centroid, the proportion of

the bending moment taken by the steel and by the tensile stresses of the concrete may be determined. If now the total vertical shear V be separated into two parts which are proportional to these moments, respectively, the part V' taken by the steel may be used in equation (18), and the part V'' taken by the tension of the concrete may be used in a similar way by considering the distance to the centroid of the tension in the concrete. Fig. 1 (b) illustrates the distribution of the vertical shear in this case.

Although the presence of tension in the concrete modifies the amount and distribution of the shearing stresses, equation (18) will be used in the discussion of the tests of beams having the bars straight throughout, since it is simpler to neglect the tension and since the results are satisfactory for the purpose of comparison.

8. *Diagonal Tension in Concrete in Beams without Web Reinforcement.*—In the flexure of a beam various stresses are set up in its web. Besides the bond and the horizontal and vertical shearing stresses already discussed, tensile and compressive and shearing stresses exist in every diagonal direction. In determining the resisting moment, only the horizontal components of these are taken. When there is no metallic web reinforcement, all the diagonal stresses are taken by the concrete. By the analysis of combined shear and tension given in text books on resistance of materials, the value of the maximum diagonal tensile unit-stress is found to be

$$t = \frac{1}{2}s + \sqrt{\frac{1}{4}s^2 + v^2} \dots\dots\dots (19)$$

where t is the diagonal tensile unit-stress, s is the horizontal tensile unit-stress existing in the concrete, and v is the horizontal or vertical shearing unit-stress. The direction of this maximum diagonal tension makes an angle with the horizontal equal to one-half of the angle whose cotangent is $\frac{s}{v}$

If there is no tension in the concrete this reduces to

$$t = v \dots\dots\dots (20)$$

and the maximum diagonal tension makes an angle of 45° with the horizontal and is equal in intensity to the vertical shearing stress at that point.

It is seen that the value of the diagonal tensile stress depends

upon the tensile stress in a horizontal direction at the given point as well as upon the amount of the horizontal and vertical shearing stresses there developed. If $s = 0$, $t = v$, and the diagonal tension has this value in a direction 45° from the horizontal. If $s = v$, $t = 1.62v$ and the direction of the maximum stress makes an angle of $31^\circ 43'$ with the horizontal. If $s = 2v$, $t = 2.4v$ and the direction of the maximum stress is $27^\circ 30'$. It may be said that in the ordinary reinforced concrete beam the value of t probably varies from one to two times v , the vertical shearing unit-stress.

It is evident that the value of the diagonal tension is generally indeterminate. No working formulas are available. For this reason it is the practice, now becoming nearly universal, in beams without web reinforcement to calculate the value of the vertical shearing unit-stress v , and to use this as the measure or means of comparison of the diagonal tensile stress developed in the beam; with the understanding, of course, that the actual diagonal tension is considerably greater than the vertical shearing stress. It will be found that the value of v developed in beams will vary with the amount of reinforcement, with the relative length of the beam, and with other factors which affect the stiffness of the beam.

9. *Diagonal Tension and Bond Failures in Beams without Web Reinforcement.*—It is evident that the tensile strength of the concrete may limit the strength of a reinforced concrete beam and that under certain conditions diagonal tension may be the cause of failure. These failures will occur in the outer parts of the length of the beam. The difference between the condition of tension in the concrete at this portion of the beam and that at the points of maximum bending moment should be noted. In the latter place the concrete will have failed in tension from the bottom of the beam to some point not far below the neutral axis, and under this condition tension in the concrete has little effect upon the resisting moment. This section is likely to be at a point in the beam where the shearing stresses are low. Failure by diagonal tension comes at a point in the beam where the longitudinal tensile stresses are less and the vertical shearing stresses are greater. The tensile strength of the concrete in beams without web reinforcement forms the principal source of resistance to diagonal tension.

Failure by diagonal tension (commonly called shear failure) generally starts from a tension crack which has formed at the bottom of the beam and extended to the level of the reinforcement or to a point somewhat above. This crack then branches in a diagonal direction, running in a direction away from the support. If the diagonal crack starts from a point some distance above the level of the reinforcement, as it may in a deep beam, it will also run diagonally downward in the prolongation of the general diagonal line. Failure by diagonal tension is generally sudden, as is the case when concrete is tested in tension and as is found in tests of unreinforced concrete beams. A variation from this is found when part of the shear is carried forward by the stiffness of the rods, and the failure is slower and progressive. Fig. 2 gives typical representations of a failure by diagonal tension in beams having all the bars straight. In the second case the reinforcing bars are finally split off from the concrete above.

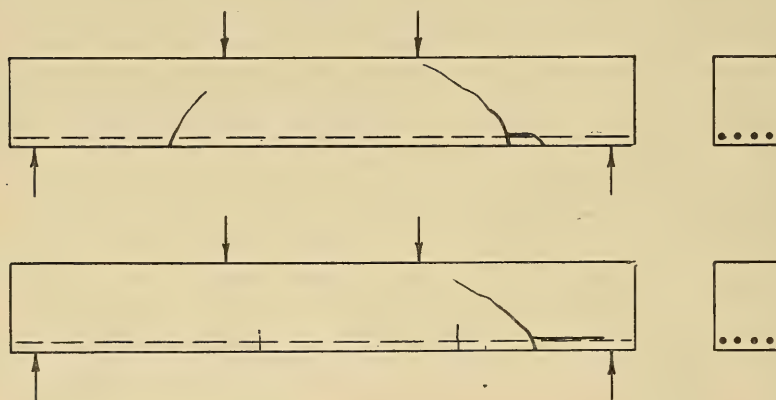


FIG. 2. POSITION OF CRACKS IN DIAGONAL TENSION FAILURES.

10. *Phenomena of Beam Action with Bars Bent Up.*—When bars are bent up in the outer portion of the beam length, the distribution and the amount of the shearing stresses and bond stresses are materially different from the foregoing. The analysis of the stresses becomes more complex; a determination of their actual value is impracticable. However, a study of the phenomena of failure and of the conditions which exist and of their variation from the conditions ordinarily assumed in the analysis of beams

with all bars horizontal, will serve a useful purpose. Comparisons may be made by means of the formulas for bond, shear, and diagonal tension already referred to.

The case where all bars are bent up at the same point will first be taken up. Assume the method of loading used in the tests, that of placing the loads at the one-third points of the span length. In Fig. 3 (a), G is the support and F the load point where one-half of the load is applied. Take any section AB at some point between the support and the load point. If we assume that the ordinary conditions of beam action exist, it will be seen that the concrete will break in tension from some point B at the bottom of the beam up to C, and with an increase of the load applied and with the increased tension and stretch in the reinforcing bar this tension crack will extend upward. With the method of loading assumed, neglecting the influence of the weight of the beam, the bending moment will increase uniformly from the support to the load point, being zero at G and a maximum at the section through F. For a beam having the reinforcement straight throughout, the common assumption of beam action would show the stress in the reinforcement to be decreasing uniformly from the load point to the support, changing from a maximum at the section through F to a zero stress at G.

With the bars bent up a different distribution of the stresses exists. We may obtain an idea of the stress at an intermediate point C, if we neglect tension in the concrete, by considering d as the distance from A to C, substituting in equation (13), p. 6, and multiplying the resulting value by $\sec \alpha$, where α is the angle which the reinforcing bar makes with the horizontal at the given point C. It is apparent that the stress in the reinforcement at C must be considerably greater than when the reinforcing bars are straight throughout. It is even conceivable, if the ordinary assumptions held, that the stress in the bars might be nearly constant to the end of the bar, as would be the case in a parabolic truss. This can not be the case in a reinforced concrete beam, however, for the tensile strength of the concrete at the bottom of the beam operates to take the greater part of the tension which goes to make up the bending moment for sections toward the end of the beam; and besides, the variation from beam

action near the end of the beam (due to the distribution of the upward pressure from the support in an oblique direction and to the existence of arch action) changes the stress distribution in sections near the end of the beam. This much is brought out, however, that if the tensile stress in the steel remained uniform to the end of the bar, the bond stress required at the end of the bar would be enormous.

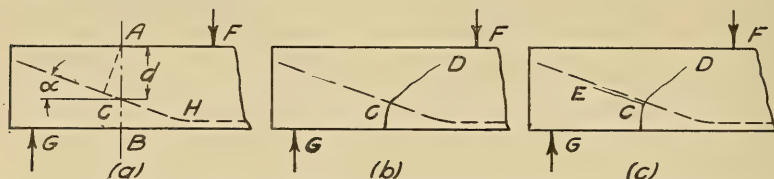


FIG. 3. DIAGONAL TENSION FAILURE IN BEAMS WITH ALL BARS BENT UP.

The presence of tensile strength in the concrete makes the above condition an impossible one; high bond stresses must also exist at certain points. Consider that at the section through C the concrete has failed in tension below the bar and that at some section nearer the support the tensile strength of the concrete is effective down to the bottom of the beam. It is evident that at the latter section the stress in the bar must be relatively small (since it is here so close to the neutral axis of the beam). The change in stress in the bar between the two sections must then be considerable and hence the bond developed on the surface of the bar will be very great, much greater than is given by the formulas as ordinarily applied. Of course, it is probable that under the actual conditions of stress the cross-section of the beam will become distorted and a plane section before bending will not remain a plane section after bending; this adds to the indeterminateness of the phenomena.

It is evident that any modification of the distribution of the tensile stress throughout the length of the reinforcing bar and of the bond between the steel and concrete at different cross-sections must be accompanied with a change in the amount and distribution of shearing stresses and diagonal tension. Where the bond stress is small, the shearing unit-stress will be low, and any condition which increases the bond stress must also increase the shearing stress and hence also the diagonal tension.

With the reinforcing bar in a diagonal direction, part of the diagonal tension will be taken by the bar. On the other hand, the formation of a vertical crack, as at C in Fig. 3 (b), gives opportunity for the starting of a diagonal crack, and the diagonal crack is likely to form from such main vertical cracks. Whether bond or diagonal tension will be the ultimate cause of failure is, then, a subject for investigation. Another condition which enters into the action of bars with reinforcement bent up in this way is the effect of the sudden change in stiffness at the point C after the vertical and diagonal crack has formed to and beyond C. It will be seen in Fig. 3 (c) that the portion of the beam to the left of C is stiffer than that at C, and with increase of bending at C a tendency to split the concrete exists. As a result the concrete may split along the reinforcing bar from C to E as shown in the figure, and this may be the manner of failure.

When part of the bars are bent up and part remain straight, the stresses are distributed in still another way. In Fig. 4 (a) consider that part of the bars (say one-half) are bent up along IK, and that the remainder run straight toward or past the support G. Consider that at the section FH both sets of bars are at the same level and that they have the same tensile stress. Away from I toward the end of the beam, the tensile stress in the upper bars will be less than that in the lower group. If we assume that a plane section before bending remains a plane section after bending the relative deformation in the two sets of bars at the section AB may be represented as at (b) in Fig. 4. Likewise, the ratio of the tensile stresses developed at C and B (shown at (c)) may be represented by $\frac{f''}{f'} = \frac{y''}{y'}$, where f'' and f' are the tensile unit-stresses in the bars at C and B, respectively, and y'' and y' are the distances shown in the diagram. It appears, then, that the stress in the rod at B must take the greater part of the bending moment and hence that the stress in the bar at B will be correspondingly greater than that in the bar at C. As the tensile stress in the bent rod has decreased a relatively large amount between the points I and C, the bond stress developed in this portion of its length must be higher than given by the ordinary equation for bond stress; and from C to K the bond stress must be correspondingly less than the calculated value. In the

lower bars the stress at B may not be much less than at I, since at B it is taking the larger part of the bending moment, and hence the bond stress developed in the portion IB will be less than that found by the formula, while beyond B and toward G the decrease in stress will be rapid and the bond stress developed will be correspondingly greater. Again it may be noted that the assumption that a plane section before bending remains a plane section after bending evidently is not exactly true, and the distortion which takes place in the concrete adds to the difficulties of determining

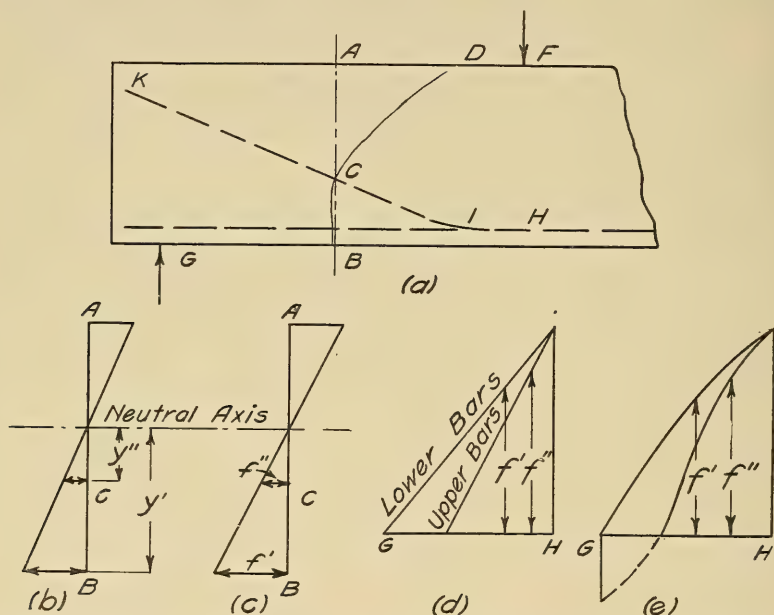


FIG. 4. DIAGONAL TENSION FAILURE IN BEAMS WITH PART OF THE BARS BENT UP.

the stresses. Fig. 4 (d) may be taken as illustrative of the change in f in the two sets of bars between the load point and the support under the assumption that ordinary beam action prevails, and Fig. 4 (e) as representative of the stresses with the conditions of distortion prevailing.

It would seem, then, that the bending up of the bars results in greater vertical shearing stresses between I and C, and that a large part of the diagonal tension developed here will be taken by

the bent reinforcement; and also that in the lower set there is less bond developed between I and B and hence that the diagonal tension throughout this portion is less than might otherwise be expected. If, now, another part of the bars were bent up at B a condition would be developed similar to that in IC. It is evident that it is not easy to determine the distribution of the diagonal stresses among the several bars, and this is especially true if they are not rigidly connected with each other.

When the main reinforcing bars run straight through the beam and diagonal reinforcement is fastened to them, or where a part of the bar is sheared and bent up, as in the Kahn bar, still another condition exists. Here the conditions of tensile stresses in the main reinforcing bars approximate those given by the ordinary beam assumptions and formulas, (modified for tension in the concrete), if the amount of metal is uniform or nearly uniform throughout the length of the beam. Most of the diagonal tension may be considered finally to be taken by the diagonal reinforcement, if its sectional area and bond resistance are sufficient and if the diagonals are properly distributed in the beam. Use V as the amount of total vertical shear at the given section which is considered to be carried through the diagonal reinforcement. The proportion which this V bears to the actual total vertical shear at the given section may not be known and possibly may have to be estimated from our knowledge of the results of tests. When the diagonals make an angle of 45° with the horizontal, the assumption that the direction of the resulting diagonal compressive stress is at an angle of 45° with the horizontal will not result in material error, particularly if the bars are bent up at intervals longitudinally equal to d' or $\frac{1}{2}d'$. For these conditions the amount of the diagonal stress finally taken by the diagonal reinforcement may be estimated to be $0.707 \frac{Va}{d'}$, where a is the distance longitudinally between diagonals. This is with the provision, of course, that the diagonals are sufficiently close together and that they are securely fastened to the longitudinal bars.

Generally speaking, then, the distribution of web stresses in beams with bars bent up is more or less indeterminate. A special study of the stresses may be made for any given arrangement of bars, and recourse must be had to experimental results for each case. In this bulletin instead of attempting to calculate the actual stresses, the values given in the tables and in the discussion will be those obtained by equations (17) and (18), using for d the

full depth from the top of the beam to the reinforcement as it exists at the middle of the beam; and the values so found will be termed the *nominal* shearing stress and the *nominal* bond. These nominal values will be used in comparison of results obtained with the different arrangements of bars, but it must be expressly understood that they do not represent actual stresses in the reinforcement or in the concrete.

11. *Web Reinforcement by Vertical Stirrups.*—The use of vertical stirrups to take a part or all of the vertical component of the web stresses is quite common. These stirrups are U-shaped or looped and enclose the longitudinal reinforcement and generally are not fastened to it. The following assumed conditions will be discussed in order: (1) the assumption that all the vertical component is taken by the stirrups; (2) the condition which exists before the concrete fails in diagonal tension; and (3) the conditions probably existing after the concrete has so failed.

(1). In Fig. 5 (a), the spacing of the stirrups is given as a . If a diagonal crack has formed, the condition shown in Fig. 5 (b) may exist. Following the line of reasoning used in establishing equation (18), p. 9, and assuming that the diagonal compressive stress makes an angle of 45° with the horizontal whatever may be the spacing of the stirrups, the vertical component to be taken by the stirrups per unit of length of beam is seen to be $\frac{V}{d}$. The total stress taken by one stirrup then is

$$P = \frac{Va}{d} \dots\dots\dots (21)$$

It is assumed that enough vertical deformation has taken place to develop this stress, and that the stirrups are close enough together to make this form of reinforcement effective. It will be seen that the assumed action corresponds in some ways with the action of a truss having the line of the upper chord at the centroid of the compressive stresses.

(2). Until the concrete web has failed in diagonal tension and diagonal cracks have formed there must be little vertical deformation at the plane of the stirrups, so little that not much stress can have developed in the stirrups. Besides, at this stage the diagonal tension must be expected to be distributed along the length of the beam and not to be concentrated at the stirrups. It is evident, then, that until the concrete web fails in diagonal tension little stress is taken by the stirrups.

(3). After the diagonal cracks have formed, the lower part of the concrete web becomes ineffective in resisting diagonal tension. The stress may be considered as delivered to the stirrup at B in Fig. 5 (b) through the cantilever projection $C'B'E$. There is still integrity of structure in the upper portion of the section CD or $C'D'$, and since there is not freedom of movement at the compression centroid it seems reasonable to consider that not the full effect is transmitted through the cantilever portion $C'B'E$ to B. The experiments seem to confirm this view. It will be convenient in the general discussion to consider that part of the vertical shear V is carried down through the cantilever $C'B'E$ to B, and part passes through the compression portion.

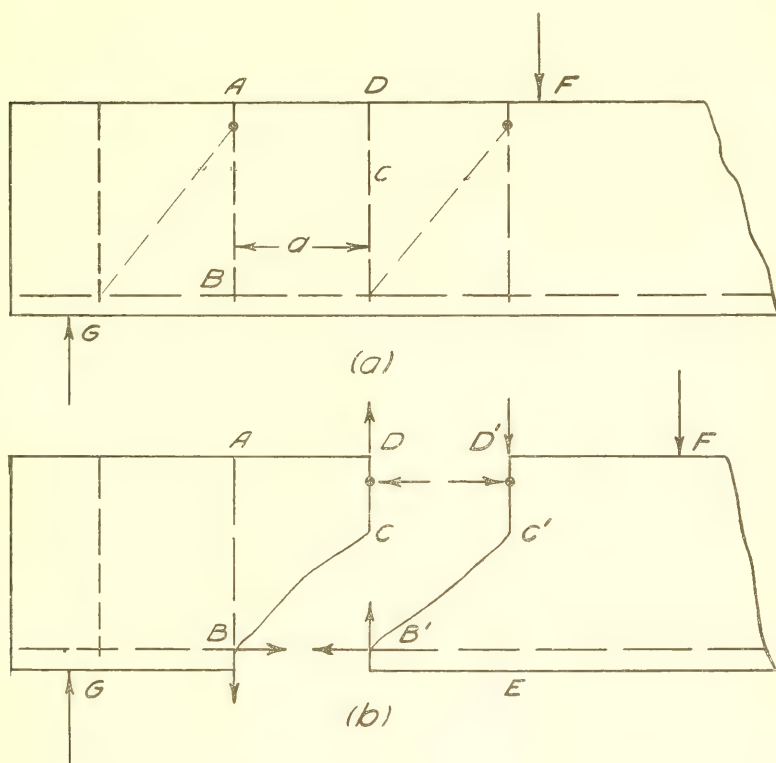


FIG. 5. ACTION OF FORCES IN BEAMS WITH STIRRUPS.

However, as the proportion taken by the stirrups is unknown, it will be assumed in the calculations in this bulletin that all of the shear is taken by the stirrups. This use of equation (21) is somewhat empirical, but it makes a common basis of comparison.

The distribution of bond stresses developed on the surface of the stirrups is indeterminate. Evidently it must not be expected that tension will be transferred to the concrete until the compression area of the beam is reached, or until a point but little below is reached. In the calculation for bond in stirrups in this bulletin, the bond surface of the stirrups for a depth of beam equal to $0.6d$ will be arbitrarily assumed.

II. MATERIALS, TEST PIECES, AND METHOD OF TESTING.

12. *Materials.*—The materials used in making the test beams were similar to those used in the reinforced concrete beams described in Bulletin No. 4 and Bulletin No. 14; they may be considered to be representative of the best materials used for this class of work in this section of the country. The stone, sand, and much of the cement were bought in the open market. The Universal portland cement, used in some of the beams, was furnished by the makers. The mild steel rods used for reinforcement were furnished by the Illinois Steel Co. The corrugated bars were supplied by the St. Louis Expanded Metal and Corrugated Bar

TABLE 1.
MECHANICAL ANALYSIS OF STONE.
Series of 1908.

Size of Mesh inches	Per cent Passing
1	100
$\frac{3}{4}$	93
$\frac{1}{2}$	64
$\frac{3}{8}$	41
$\frac{1}{3}$	22
$\frac{1}{5}$	4
$\frac{1}{10}$	2

Co. A form of reinforcement known as the Cummings' unit frame was furnished by Mr. Robert Cummings of Pittsburg, Pennsylvania.

TABLE 2.
MECHANICAL ANALYSIS OF SAND.

Sieve No.	Separation Size inches	Per cent Passing	
		1907	1908
3	—	—	98
5	.174	100	87
10	.091	81	63
18	.043	53	37
30	.027	35	20
40	.019	24	11
50	.013	14	4
74	.009	7	2
150	—	2	1

TABLE 3.
TENSILE STRENGTH OF CEMENT.

Ref. No.	Ultimate Strength, lb. per sq. in.							
	Chicago AA Cement				Universal Cement			
	Age 7 days		Age 28 days		Age 7 days		Age 28 days	
	Neat	1-3	Neat	1-3	Neat	1-3	Neat	1-3
Series of 1907								
1	786	230	811	265	410	187	680	370
2	683	145	851	235	470	200	670	330
3	760	218	861	225	360	120	560	360
4	786	186	760	270	405	145	570	290
5	866	215	965	265	320	195	600	295
6	815	211	935	287	310	180	620	310
Av.	783	201	864	258	379	171	617	326
Series of 1908								
1	559	145	707	247	565	244	764	319
2	732	192	857	318	809	248	885	336
3	665	175	779	266	728	232	776	285
4	811	227	833	307	699	242	754	292
5	666	182	792	284	702	229	763	315
6	693	191	781	283				
7	719	206	767	303				
Av.	692	188	788	287	701	239	788	309

These tests were made with standard Ottawa sand. Tests from one sample of the 1908 cement on briquettes made with natural sand gave a higher strength than did the briquettes made with the standard sand.

Each of the above values is the average of 5 briquettes

TABLE 4.
MECHANICAL ANALYSIS OF CEMENT.

1907			1908		
Sieve No.	Per cent Passing		Sieve No.	Per cent Passing	
	Universal	Chicago AA		Universal	Chicago AA
74	98.6	96.4	74	99.3	98.2
100	96.2	91.2	100	98.5	95.0
200	80.9	69.8	200	90.1	80.6

TABLE 5.
TENSION TESTS OF STEEL.
Series of 1907.

Beam No.	Nominal Size inches	Yield Point lb. per sq. in.	Ultimate Strength lb. per sq. in.	Per cent Elongation in 8 in.	Beam No.	Nominal Size inches	Yield Point lb. per sq. in.	Ultimate Strength lb. per sq. in.	Per cent Elongation in 8 in.
211.1	$\frac{3}{4}$	36 800	56 100	33	416.5	$\frac{1}{2}$	40 700	52 500	29.7
211.2	$\frac{1}{2}$	39 800	55 900	29	416.6	"	40 000	54 800	30
212.1	"	39 700	55 700	29	511.1	"	40 100	54 900	31.5
212.2	"	40 200	56 400	30.5	511.2	"	37 400	53 500	29.5
212.5	"	39 600	54 700	29	512.1	"	39 600	58 900	28
212.6	"	39 900	55 700	28.5	512.2	"	37 300	54 800	29
613.2	"	39 300	54 400	29	531.1	$\frac{5}{8}$	37 400	51 900	31.5
616.2	"	41 000	55 900	29.3	531.2	"	38 400	56 500	31.5
417.5	$\frac{5}{8}$	38 700	54 900	31.5	513.1	$\frac{1}{2}$	38 800	53 400	29.5
417.6	"	39 300	55 700	31	551.2	"	39 400	57 200	28.5
611.1	"	37 200	54 600	30.5	553.1	"	63 200	97 100	9.5
611.2	"	40 500	53 800	30	231.5	"	60 800	94 700	8.5
612.1	"	37 000	51 300	32	232.5	"	52 000	82 300	11
612.2	"	39 200	53 200	31.5	232.6	"	57 200	89 300	10
613.1	"	38 400	54 800	32	233.5	"	58 500	88 100	14.2
616.1	"	38 700	59 600	31.5	233.6	"	64 400	102 500	7.2
617.1	"	38 300	52 500	31.5	233.7	"	57 600	90 900	10.5
213.1	$\frac{1}{2}$	40 700	53 000	30	235.2	"	53 100	86 300	11
213.2	"	39 100	56 000	29.5	235.5	"	64 000	101 500	7
414.5	"	38 700	54 100	30.2	235.6	"	60 000	98 000	11
414.6	"	39 200	54 500	29.2	241.5	"	51 700	84 500	12
214.1	"	39 900	53 500	29.5	221.1	$\frac{3}{4}$	39 000	56 400	32
214.2	"	39 200	52 500	31.5	221.2	"	43 100	64 300	29
415.1	"	40 200	54 000	30.2	223.6	$\frac{1}{2}$	39 100	56 300	29.5
415.2	"	40 100	55 800	29.5	225.5	$\frac{3}{4}$	37 500	57 000	31
415.5	"	40 400	55 800	30.2	227.5	"	40 500	58 000	30
415.6	"	40 400	57 300	28.2	227.6	"	38 500	60 000	30.5
215.1	"	42 100	56 200	30	228.5	"	38 700	61 500	31
215.2	"	43 900	58 100	30.5	228.6	"	39 800	63 100	30

TABLE 5—Continued.

TENSION TESTS OF STEEL.

Series of 1908.

Beam No.	Nomi- nal Size inches	Yield Point lb. per sq. in.	Ulti- mate Str'ngth lb. per sq. in.	Per cent Elonga- tion in 8 in.	Beam No.	Nomi- nal Size inches	Yield Point lb. per sq. in.	Ulti- mate Str'ngth lb. per sq. in.	Per cent Elonga- tion in 8 in.
210.1	$\frac{1}{2}$	40 000	61 200	28.5	336.2	$\frac{1}{2}$	40 000	61 000	27
210.3	"	38 700	59 000	29.3	336.2	"	41 600	63 000	27
210.2	"	41 100	61 800	29	336.2	"	40 300	60 700	28
210.2	"	42 200	62 700	28	336.3	"	40 300	60 600	27.4
343.2	"	40 100	59 900	28.6	213.3	"	39 100	61 500	29.5
341.3	"	40 300	60 700	29	213.4	"	39 000	60 100	31
341.3	"	41 000	59 800	29	351.2	"	40 000	61 000	27.7
341.3	"	41 600	64 000	26	214.3	"	39 800	62 500	26.5
341.3	"	40 300	63 500	29	214.4	"	39 400	59 300	28.5
211.4	"	40 200	62 600	28.5	214.5	"	41 100	61 800	28.0
211.5	"	40 200	61 300	29	352.1	"	41 900	62 200	28.0
331.1	"	42 500	63 600	27	352.2	"	40 900	61 300	28
331.3	"	40 800	60 700	29.3	353.2	"	39 200	59 500	28
332.1	"	40 500	57 900	26	351.1	"	40 800	60 900	28.5
217.0	"	39 700	62 500	29	215.3	"	39 600	59 500	29.5
217.1	"	39 900	63 800	26.5	215.4	"	40 500	61 400	27
217.3	"	39 900	61 600	29	361.1	"	39 900	61 300	27.5
217.5	"	40 900	60 700	29.5	362.1	"	40 200	61 000	28
217.2	"	38 200	61 100	30	362.2	"	40 300	61 200	27.5
217.4	"	41 200	62 000	28	363.2	"	40 300	60 300	28
217.4	"	40 000	61 000	29	361.2	"	40 400	61 300	27.5
218.1	"	40 800	61 200	29	363.1	"	40 200	61 000	30.2
218.1	"	39 300	62 300	29	221.3	$\frac{3}{4}$	37 900	58 300	32
218.2	"	40 400	61 700	29.5	221.4	"	37 100	60 100	30.5
218.3	"	40 900	60 600	27.5	223.2	$\frac{1}{2}$	41 300	61 000	28.5
218.3	"	40 400	60 500	28.5	223.3	"	40 900	60 500	29
254.1	"	40 400	61 800	29.4	229.1	"	41 100	60 000	28.3
252.5	"	52 800	88 600	15	229.2	"	41 500	63 300	28
252.6	"	51 400	84 800	18	229.2	"	40 600	61 200	27
252.7	"	45 600	77 900	19.5	229.4	"	39 400	61 200	27.5
253.5	"	48 700	83 000	16.2	229.5	"	41 500	61 100	27.5
253.6	"	49 400	80 000	19	229.7	"	40 200	60 500	29
219.1	"	39 300	59 500	27.7	229.8	"	40 300	60 500	29
219.2	"	40 600	61 700	27.3	229.8	"	37 500	61 200	27
255.1	$\frac{3}{4}$	38 000	60 600	33	229.8	"	37 600	59 400	27
255.2	"	41 000	60 800	28.6	233.3	"	40 500	60 600	26.5
220.5	1	39 200	64 500	30	233.4	"	49 400	80 800	21.5
220.1	$\frac{3}{4}$	36 900	63 100	35.5	241.1	"	51 000	84 000	13
226.2	"	36 500	60 300	30	241.2	"	47 500	79 100	20.3
256.1	"	38 600	61 400	29.5	241.7	$\frac{3}{4}$	42 200	60 700	33
256.2	"	38 100	61 100	31.2	241.8	"	45 000	62 600	30
253.1	"	53 400	89 100	11.5	243.3	$\frac{1}{2}$	50 100	81 800	18
336.2	$\frac{1}{2}$	40 300	61 300	29	243.4	"	36 100	56 400	26

Stone. The stone was a good quality of crushed limestone from Kankakee, Illinois, ordered screened through a 1-in. and

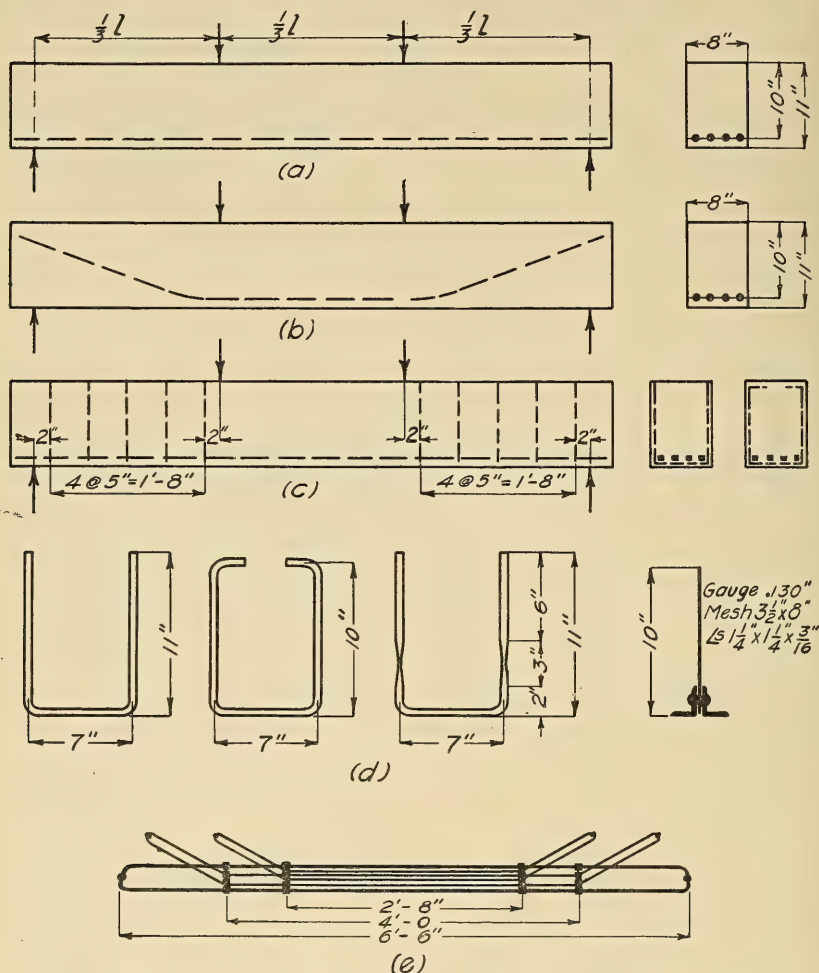


FIG. 6. DETAILS AND ARRANGEMENT OF REINFORCEMENT.

over a $\frac{1}{4}$ -in. screen. It contained about 50 % voids and weighed about 81 lb. per cu. ft. In the determination of the voids in both stone and sand, the material was poured slowly into water to obviate the retention of air. Table 1 gives the average of the tests of 19 samples of the stone used in 1908.

Sand. The sand was of good quality, sharp, well graded, and

generally clean; it weighed 104 lb. per cu. ft. loose. Table 2 gives the results of mechanical analyses of this sand.

Cement. Tests of the two brands of cement used for the two years are given in Table 3. Table 4 gives analyses of the fineness.

Concrete. Men accustomed to mixing concrete and making test beams were employed in the work. Care was taken in measuring, mixing, and tamping to secure as uniform a concrete as possible. All materials were proportioned by loose volume, and weights were taken as a check on the measurement. The mixing was done with shovels by hand. The sand and cement were first mixed dry; the stone, which had previously been thoroughly moistened, was added and the mass then turned until of a uniform appearance. Water was then added in such proportion as to give a fairly wet mixture. The mass was again turned until thoroughly mixed.

Steel. The steel reinforcing bars consisted of plain round rods and deformed bars. The round rods were open hearth mild steel. The deformed bars were square corrugated bars with "new style" corrugations. Two grades of the deformed bars were used, one of mild steel for some of the stirrups and one of high elastic limit. Test pieces were cut from all the reinforcing bars used. Table 5 gives the results of the tests. In those beams which failed by tension in the steel individual results are recorded. In those in which the tensile strength of the steel was not developed the average of all tests for the several sizes is given. Tests of Cummings' reinforcement showed the yield point of the steel to be about 44 000 lb. per sq. in.

13. *Test Beams.*—In all the tests herein described, the cross-section of the beams was 8 x 11 in., the center of the longitudinal reinforcement being placed 10 in. below the top surface throughout at least the middle third of the length of the beam. The span length varied from 6 ft. to 12 ft. The total length of the beam was 6 in. more than the span length in the beams of 6 ft. span length, and 12 in. more in the 12 ft. lengths. In a large proportion of the beams, the reinforcing bars were straight from end to end. In the beams with bars bent up, the bending of the bars began at a point about 3 in. outside the load point and passed diagonally in a slightly curved line to a point $2\frac{1}{2}$ in. or 5 in. from the top of the

beam over the supports. The general position and disposition of the bars are shown in Fig. 6, and the details are given in Tables 8, 11, and 13. The arrangement of nuts and washers at the end of the bars and of hooked ends, as used in some of the beams, is shown in Fig. 14 and 18.

Included in the beams with bars bent up are those made with the Cummings' unit frame. This reinforcement (see Fig. 6) consisted of two $\frac{5}{8}$ -in. round rods carried straight to a point beyond the supports and there bent and welded to form a loop near the end of the beam, one pair of $\frac{3}{8}$ -in. round rods bent up diagonally at the one-sixth point and looped and welded near the top of the beam, and another pair of $\frac{3}{8}$ -in. round rods bent up at a point 8 inches from the one-sixth point. These rods were connected laterally with the $\frac{5}{8}$ -in. rods at the points of bending by means of steel plates.

In beams with stirrups, the stirrups were U-shaped and passed under the longitudinal bars and extended to the top of the beam. In some beams the stirrups were left too close to the side of the beam to be effective. Fig. 6 shows the shape of the stirrups and their general spacing, and Fig. 16, 17 and 18 show the arrangement for individual beams. The size and spacing of the stirrups are given in Table 13. The stirrups were placed in the outer thirds of the beams, except that in Beams No. 229.1, 229.2, 229.5 and 229.8 stirrups were placed in one end only. The stirrups in Beams No. 222.5, 222.6, 224.5 and 224.6 were tapered to a reduced section, as shown in Fig. 6. The longitudinal reinforcing bars were straight, except in Beam No. 229.8, in which two bars were bent up in the end not having stirrups.

Two beams were reinforced with two small angles riveted together and holding between them a web of expanded metal which extended up to the top of the beam. This form of reinforcement was used by the company furnishing it. The arrangement is shown in Fig. 6.

For general data of beams see Table 8, p. 34, Table 11, p. 35, and Table 13, p. 62.

14. *Making of the Beams.*—The beams were made in a manner similar to that described in Bulletin No. 14. They were built directly on the concrete floor of the laboratory with a strip of building paper beneath the forms. The forms were generally removed

after seven days, but the beams were not moved until the date of test.

15. *Minor Test Pieces.*—In many cases tests were made on 6-in. cubes from the batch of concrete used in the beams. The results are given in Table 6. In addition to this a flexure test was made of a plain concrete control beam or check beam to give a means of judging of the resistance of the concrete to tension. These control beams were 6 x 8 x 40 in. and were tested with a 3-ft. span and one-third point loading. In testing, they were placed on a wooden base, so arranged as to insure a good distribution of the loads and pressures across the width of the beam, the wood acting as a cushion and permitting adjustment for any warped condition of the beam. The results of these tests are given in Tables 9, 10, and 12.

16. *Storage.*—The beams were left on the floor of the mixing

TABLE 6.
COMPRESSION TESTS OF 6-IN. CUBES.
Series of 1907.

No.	Kind of Concrete	Age at Test days	Maximum Load lb. per sq. in.	No.	Kind of Concrete	Age at Test days	Maximum Load lb. per sq. in.
211.1	1-1½-3	352	5080	224.5	1-2-4	303	1960
211.1	"	"	5340	224.6	1-2-4	267	2110
211.1	"	"	3640	225.7	1-2-4	332	3730
211.1	"	359	4310	225.7	"	336	3400
212.5	1-2-4	357	3610	225.7	"	"	3240
212.5	"	359	3680	227.5	1-2-4	313	4170
212.5	"	"	3200	227.5	"	316	3842
212.5	"	"	3100	227.5	"	318	4110
212.5	"	"	2870	235.6	1-5-10	334	1165
214.1	1-4-8	342	1370	235.6	"	"	1150
214.1	"	344	1260	241.6	1-2-4	317	3300
217.5	1-2-4	303	1960	241.6	"	325	3120
221.1	1-2-4	334	2570	241.6	"	"	3030
221.6	1-2-4	323	2530	271.6	1-2-4	317	3300
222.6	1-2-4	267	2110	271.6	"	325	3120
223.6	1-2-4	313	4170	271.6	"	"	3030
223.6	"	316	3842	417.6	1-2-4	303	2420
223.6	"	318	4110	521.5	1-2-4	323	2530

TABLE 6—*Continued.*
 COMPRESSION TESTS OF 6-IN. CUBES.
 Series of 1908.

No.	Kind of Concrete	Ages at Test days	Maximum Load lb. per sq. in.	No.	Kind of Concrete	Age at Test days	Maximum Load lb. per sq. in.
210.1	1-1-2	65	4045	217.5	1-2-4	75	1537
210.1	"	"	3995	217.5	"	"	1692
210.1	"	"	3835	217.5	"	"	1731
210.2	1-1-2	70	5715	218.2	1-2-4	59	2583
210.2	"	"	4670	218.2	"	"	2650
210.2	"	"	5180	218.2	"	"	2778
210.3	1-1-2	63	3520	218.3	1-2-4	60	2700
210.3	"	"	3560	218.3	"	"	2240
210.3	"	"	3620	218.3	"	"	2620
211.4	1-1-3	65	3275	218.4	1-2-4	68	1970
211.4	"	"	3080	218.4	"	"	1725
211.4	"	"	3590	218.4	"	"	1870
213.3	1-3-6	65	1722	219.2	1-2-4	59	2583
213.3	"	"	1752	219.2	"	"	2650
213.3	"	"	1590	219.2	"	"	2778
213.4	1-3-6	65	1130	220.2	1-2-4	59	2392
213.4	"	"	1034	220.2	"	"	2270
213.4	"	"	1192	220.2	"	"	2322
214.4	1-4-8	68	1297	220.5	1-2-4	67	2370
214.4	"	"	1364	220.5	"	"	2150
214.4	"	"	1255	220.5	"	"	2110
215.3	1-5-10	64	1075	221.3	1-2-4	57	2672
215.3	"	"	1075	221.3	"	"	2450
215.3	"	"	1143	221.3	"	"	2735
215.4	1-5-10	69	1227	221.4	1-2-4	62	1810
215.4	"	"	1218	221.4	"	"	1890
215.4	"	"	1080	221.4	"	"	1660
215.5	1-5-10	60	942	223.2	1-2-4	57	2672
215.5	"	"	892	223.2	"	"	2450
215.5	"	"	872	223.2	"	"	2735
217.0	1-2-4	61	2980	223.3	1-2-4	62	1810
217.0	"	"	3340	223.3	"	"	1890
217.0	"	"	2670	223.3	"	"	1660
217.1	1-2-4	15	1310	229.1	1-2-4	61	2980
217.1	"	"	1135	229.1	"	"	3340
217.1	"	"	1140	229.1	"	"	2670
217.2	1-2-4	66	2330	229.2	1-2-4	83	3140
217.2	"	"	2100	229.2	"	"	2850
217.2	"	"	2070	229.2	"	"	3250
217.4	1-2-4	69	2880	229.5	1-2-4	75	1537
217.4	"	"	2900	229.5	"	"	1692
217.4	"	"	2960	229.5	"	"	1731

TABLE 6—*Continued.*

COMPRESSION TESTS OF 6-IN. CUBES.

Series of 1908.

No.	Kind of Concrete	Age at Test days	Maximum Load lb. per sq. in.	No.	Kind of Concrete	Age at Test days	Maximum Load lb. per sq. in.
233.3	1-2-4	74	1700	331.3	1-2-4	34	391
233.3	"	"	1520	331.3	"	"	410
233.3	"	"	1410	331.3	"	"	420
240.1	1-2-4	62	2750	332.1	1-2-4	11	1420
240.1	"	"	2490	332.1	"	"	1510
240.1	"	"	2580	332.1	"	"	1620
240.2	1-2-4	59	1770	333.1	1-2-4	17	626
240.2	"	"	1810	333.1	"	"	640
240.2	"	"	1770	333.1	"	"	738
243.3	1-2-4	62	2750	342.2	1-1-2	69	3880
243.3	"	"	2400	342.2	"	"	4030
243.3	"	"	2580	342.2	"	"	3640
252.6	1-2-4	59	2392	351.1	1-4-8	202	1420
252.6	"	"	2270	351.1	"	"	1131
252.6	"	"	2322	351.1	"	"	1232
252.7	1-2-4	59	3008	351.2	1-4-8	14	828
252.7	"	"	3194	351.2	"	"	752
252.7	"	"	2964	251.2	"	"	866
253.1	1-2-4	59	2392	352.1	1-4-8	62	1490
253.1	"	"	2270	352.1	"	"	1620
253.1	"	"	2322	352.1	"	"	1610
253.5	1-2-4	65	3480	352.2	1-4-8	63	1028
253.5	"	"	3730	352.2	"	"	997
253.5	"	"	3770	352.2	"	"	1000
253.6	1-2-4	59	3008	353.2	1-4-8	68	831
253.6	"	"	3194	353.2	"	"	745
253.6	"	"	2964	353.2	"	"	884
254.1	1-2-4	67	2025	361.1	1-5-10	49	1050
254.1	"	"	2160	361.1	"	"	1040
254.1	"	"	2125	361.1	"	"	985
255.1	1-2-4	68	2360	362.1	1-5-10	60	842
255.1	"	"	2300	362.1	"	"	937
255.1	"	"	2040	362.1	"	"	832
255.2	1-2-4	61	2430	362.2	1-5-10	69	1010
255.2	"	"	2475	362.2	"	"	820
255.2	"	"	2470	362.2	"	"	862
256.1	1-2-4	68	2360	363.1	1-5-10	184	733
256.1	"	"	2300	363.1	"	"	720
256.1	"	"	2040	363.1	"	"	1005
256.2	1-2-4	61	2430	363.2	1-5-10	68	570
256.2	"	"	2475	363.2	"	"	598
256.2	"	"	2470	363.2	"	"	640
331.1	1-2-4	6	776				
331.1	"	"	834				
331.1	"	"	897				

room until the date of test, when they were removed to the Materials Testing Laboratory. They were dampened occasionally to prevent too rapid drying. The temperature ranged from 55° to 70° F. Owing to incompleteness of the heating arrangement, the temperature of the room during the early part of the period of making beams in 1907 was less uniform than could have been desired, and the beams first made probably did not have as favorable conditions for setting as would usually be the case.

17. *Method of Testing.*—In testing, the beams were loaded at the one-third points, as described in Bulletin No. 4, page 34, except that in Beams No. 229.1, 229.2 and 229.5 a single load was applied at the one-third point next to the end having the stirrups. The beams were all tested in the 200 000-lb. Olsen testing machine. The bedding of the plates and the facilities for longitudinal adjustment in the test were the same as those in the earlier tests.

Center deflections were read on all the beams. Deformations of the upper fiber and steel were measured on only part of the beams; the discussion of this part of the work is left for a later bulletin. The methods of measuring the deflections and deformations are fully described in Bulletin No. 4.

III. EXPERIMENTAL DATA AND DISCUSSION.

18. *Explanation of Tables.*—Tables 8, 11, and 13 give data of the make-up of the test beams. Tables 9, 12, and 14 contain data of the tests of the beams. The per cent of cement recorded in the tables is based upon weights and is given in terms of the aggregate (stone and sand) and not of the resulting concrete. The loads given in the tables are the loads applied by the testing machine, and do not include the weight of the beam. The load at first diagonal crack is the load noted when the first diagonal crack was observed. The stress in the longitudinal reinforcement is calculated from the bending moment, taking into account the weight of the beam. In determining the amount of the vertical shearing stress an amount was added to include the effect of the weight of the beam at about the one-sixth point, together with the loading apparatus. In the calculation of stresses, equation (13), p. 6, $M = Afd'$, was used for calculating the stress in longitudinal reinforcement. The vertical shearing unit stress was calculated with equation (18), p. 9, $v = \frac{V}{bd}$, and the bond

unit stress with equation (17), p. 8, $u = \frac{V}{mod}$. The values of d' were selected with reference to the amount of reinforcement and the modulus of elasticity of the concrete. The moduli for different mixtures and ages had been determined by another investigation, and the values of j (ratio of d' to d) were calculated by the method given in Bulletin No. 4. The values of j used are

TABLE 7.
VALUES OF j USED IN CALCULATIONS.

Kind of Concrete	Reinforcement per cent	Age, days			
		7	14	30	60
1-1-2	1.00	.825	.845	.865	.87
1-1½-3	1.00		.84		.865
"	1.65				.84
1-2-4	1.00	.80	.825	.845	.85
"	1.25				.84
"	1.30				.835
"	1.50				.83
"	1.65				.825
"	1.96				.82
"	2.20				.81
"	2.81				.80
1-3-6	1.00				.845
1-4-8	1.00		.80	.825	.84
1-5-10	1.00				.825
"	1.25				.82

given in Table 7. In all cases, whether the bars were bent up or not, and whether there were stirrups or not, the foregoing formulas for shearing stresses and bond stresses were used. The term *nominal* shearing stress is employed, as noted on page 18. The modulus of rupture of the control beams was calculated by the usual rectangular beam formula. The stress in the stirrups was calculated by the methods given on page 18. For the bond stress in stirrups the area of stirrup available was considered to be that taken by a length of stirrup equal to six-tenths of the distance from the top of the beam to the center of the reinforcing bars.

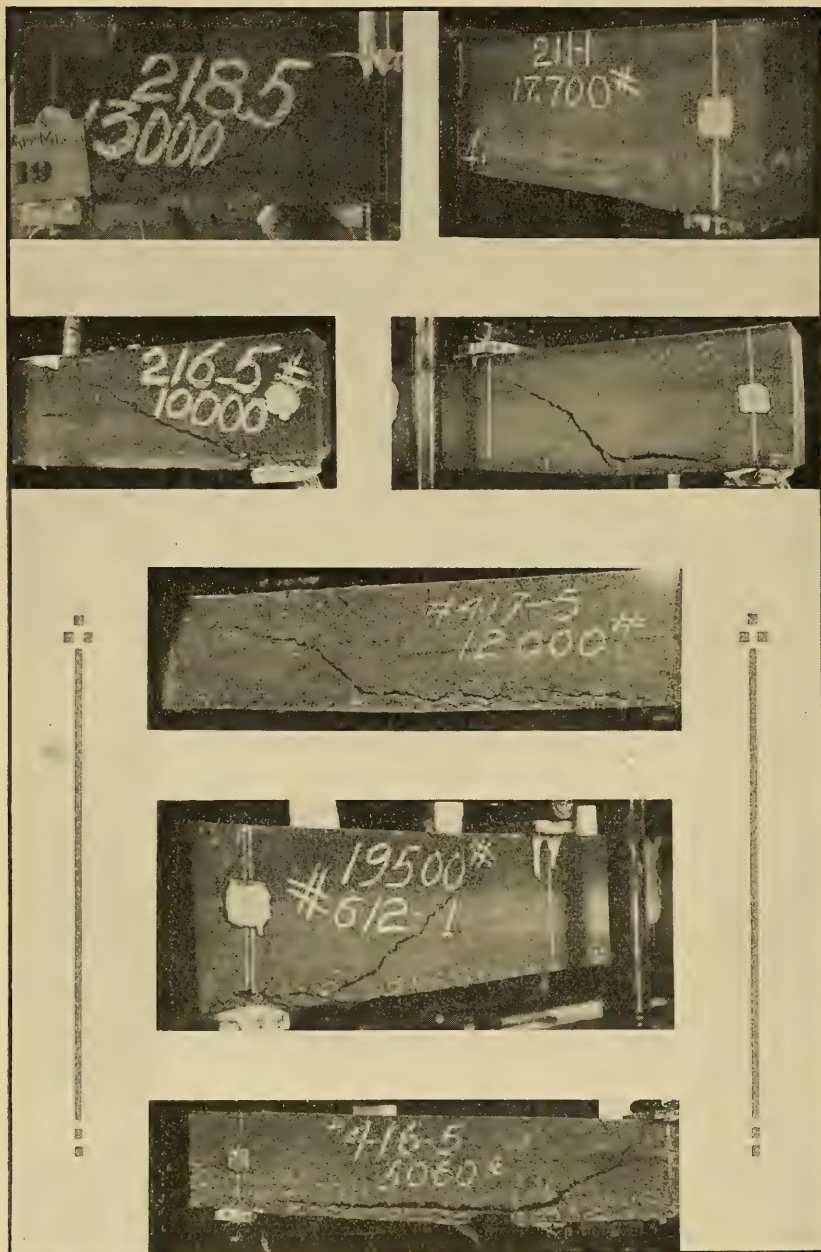


FIG. 7. VIEWS OF BEAMS AFTER TEST.

19. *Deflection Diagrams.*—Diagrams showing the center deflections of the various beams for the applied loads are given in Fig. 19 to 25 at the end of the text.

A. BEAMS WITHOUT WEB REINFORCEMENT.

20. *Beams without Web Reinforcement.*—In beams without web reinforcement which fail by diagonal tension, the first sign of approaching failure to be noted is a small diagonal crack generally found somewhat below the middle of the depth of the beam at a point about half way between the load point and the end support. Sometimes it seems to start at the level of the reinforcing bars, or is not noted until it has extended that far down, running from a vertical crack which has already formed up to that level. As the load on the beam is increased, the crack lengthens and extends from the level of the reinforcing bar diagonally upward, reaching the load point or to within a few inches of the load point at or near the maximum load carried. The angle which the crack makes with the horizontal and its position with reference to the support and load point depend upon the strength of the concrete forming the web, upon the length or slenderness of the beam, and upon the amount of reinforcement.

In one or two cases the crack extended diagonally from the support to the load point. In many beams a longitudinal crack formed along the upper level of the reinforcing bars toward the support, the bars stripping off from the concrete above by the action of vertical tension. The views in Fig. 7 show various forms of cracks in beams of the 1907 series. The characteristics of these cracks will be taken up farther on. Generally the beam finally carried a load greater than that applied when the diagonal crack was first noted, but in many of the beams no crack was noted until the maximum load was reached, and then failure along this diagonal crack was sudden and complete. In general, after the maximum load was reached, the load fell off at once, and a dead load would have produced instant failure.

The disposition of bars used in this group (no bars bent up and no stirrups) permits a comparison of the resistance of the concrete to diagonal tension without the complication which would otherwise exist. As the bars extended beyond the beam supports ($1\frac{1}{2}$ in. in the 6-ft. span, and 3 in. in the 12-ft. span), there is a some-

TABLE 8.
DATA OF BEAMS WITHOUT WEB REINFORCEMENT.
Series of 1907.

Beam No.	Kind of Concrete	Cement*		Reinforcement		Span Length feet
		Kind	Per cent	Description	Per cent	
218.5	1-1½-3	U	18.8	4 ½-in. round	0.98	6
218.6	do.	U	18.8	do.	0.98	6
211.3	1-1½-3	AA	20.6	4 ½-in. round	0.98	6
211.1	do.	AA	22.2	3 ¾-in. round	1.65	6
211.2	do.	AA	21.1	do.	1.65	6
216.1	1-2-4	AA	15.7	4 ½-in. round	0.98	6
216.2	do.	AA	14.6	do.	0.98	6
216.5	do.	U	15.9	do.	0.98	6
216.6	do.	U	10.6	do.	0.98	6
217.5 ₁	1-2-4	U	15.9	4 ½-in. round	0.98	6
217.6	do.	U	10.6	do.	0.98	6
217.5 ₂	do.	U	15.7	do.	0.98	6
212.7	1-2-4	U	15.4	4 ½-in. round	0.98	6
212.1	1-2-4	AA	16.1	5 ½-in. round	1.23	6
212.2	do.	AA	15.3	do.	1.23	6
212.5	do.	U	15.1	do.	1.23	6
212.6	do.	U	15.5	do.	1.23	6
613.2	do.	AA	13.4	do.	1.23	6
616.2	do.	AA		do.	1.23	6
251.1	do.	AA	14.6	4 ½-in. cor.	1.25	6
251.2	do.	AA	17.5	do.	1.25	6
251.3	do.	AA	14.3	do.	1.25	6
251.4	do.	AA	14.3	do.	1.25	6
417.5	1-2-4	U	15.6	4 ⅝-in. round	1.53	12
417.6	do.	U	15.6	do.	1.53	12
611.1	1-2-4	AA	15.5	4 ⅝-in. round	1.53	6
611.2	do.	AA	15.8	do.	1.53	6
612.1	do.	AA	15.4	do.	1.53	6
612.2	do.	AA	15.8	do.	1.53	6
613.1	do.	AA	15.4	do.	1.53	6
616.1	do.	AA	15.8	do.	1.53	6
617.1	do.	AA	16.0	do.	1.53	6
617.2	do.	AA	15.5	do.	1.53	6
213.1	1-3-6	AA	10.7	4 ½-in. round	0.98	6
213.2	do.	AA	10.3	do.	0.98	6
414.5	do.	U	11.2	do.	0.98	12
414.6	do.	U	13.0	do.	0.98	12
214.1	1-4-8	AA	8.4	do.	0.98	6
214.2	do.	AA	8.5	do.	0.98	6
415.1	do.	AA	8.2	do.	0.98	12
415.2	do.	AA	7.1	do.	0.98	12
415.5	do.	U	8.0	do.	0.98	12
415.6	do.	U	8.0	do.	0.98	12
215.1	1-5-10	AA	6.5	4 ½-in. round	0.98	6
215.2	do.	AA	6.5	do.	0.98	6
416.5	do.	U	6.4	do.	0.98	12
416.6	do.	U	5.7	do.	0.98	12

[Continued on page 36.]

TABLE 9.

TESTS OF BEAMS WITHOUT WEB REINFORCEMENT.

Series of 1907.

All beams failed by diagonal tension unless otherwise noted.

Beam No.	Age days	Load at First Diagonal Crack pounds	Maximum Applied Load pounds	Stress in Longi'tal Reinforcement lb. per sq.in.	Vertical Shearing Stress lb. per sq. in.	Bond Stress lb. per sq. in.	Control Beam	
							Modulus of Rupture lb. per sq.in.	Age days
218.5	14	12 000	13 000	24 500	103	139	248	14
218.6	14	12 000	14 900	27 900	114	148	248	14
211.3	61	18 000	19 100	34 300	144	194	354	61
211.1	63	18 000	29 000	31 800	222	251	331	61
211.2	67	15 000	18 300	20 200	142	160	310	57
216.1	7		9 000	18 000	71	95	223	62
216.2	7	6 000	6 000	12 300	54	73	135	10
216.5	9	10 000	10 000	19 900	86	116	202	10
216.6	8	8 900	8 900	17 800	76	103	137	10
217.5 ₂	14	11 000	11 600	22 300	94	127	202	10
217.6	14	12 000	12 000	23 100	97	131	138	10
217.5 ₁	25	12 000	14 600	27 200	114	154	147	23
212.7	60	18 100	18 100	33 400	138	186	336	77
212.1	63	19 000	20 000	29 700	155	167	228	61
212.2	59	14 000	20 900	31 000	162	175	305	57
212.5	64	14 000	15 400	23 000	121	131	221	61
212.6	61	14 000	17 000	25 300	132	143	246	57
613.2	61	18 000	18 500	27 500	144	156	280	94
616.2	63	20 000	24 700	36 500	190	205	260	94
251.1	60	19 200	19 200	28 100	149	149	291	63
251.2	60	14 500	14 500	21 300	114	114	183	61
251.3	60	12 300	12 800	18 800	102	102	238	61
251.4	60	14 000	14 000	20 600	110	110	288	61
417.5	61		12 000	30 400	100	102	229	62
417.6	63		10 500	26 800	89	91	189	63
611.1	57	16 000	19 000	22 900	149	152	293	60
611.2	61	22 400	22 400	27 000	175	178	252	55
612.1	57	10 000	19 500	23 500	153	156	339	60
612.2	61	14 000	18 450	22 200	145	148		
613.1	57	12 000	14 000	17 000	111	113	307	60
616.1	60	18 000	20 000	24 100	156	159	249	64
617.1	62	16 000	21 700	26 100	169	172	304	61
617.2	57	18 000	22 250	26 800	173	176	304	60
213.1	63	9 000	12 400	23 100	98	132	178	60
213.2	57	11 000	11 400	21 300	90	121	204	65
414.5	59		7 920	31 600	69	93		
414.6	61		8 430	33 500	72	97		
214.1	62	11 000	11 300	21 200	90	121	179	56
214.2	57	9 500	9 500	17 900	77	104	174	57
415.1	75		8 680	34 300	75	101	135	78
415.2	62		7 220	29 000	64	86	188	87
415.5	57		7 000	28 300	62	84	151	63
415.6	62		8 000	30 400	70	95	125	63
215.1	62	10 000	10 200	19 100	84	113	174	56
215.2	61	8 000	8 600	19 900	72	97		
416.5	65		5 060	21 300	49	66	109	61
416.6	70		5 830	24 100	54	73	118	63

[Continued on page 37.]

TABLE 8.—*Continued.*

DATA OF BEAMS WITHOUT WEB REINFORCEMENT.

Series of 1908.

Beam No.	Kind of Concrete	Cement		Reinforcement		Span Length feet
		Kind	Per cent	Description	Per cent	
210.1	1-1-2	AA	28.5	4 $\frac{1}{2}$ -in. round	0.98	6
210.3	do.	AA	30.7	do.	0.98	6
210.2	do.	AA	29.6	do.	0.98	6
343.2	do.	AA	29.8	6 $\frac{1}{2}$ -in. round	1.47	12
341.3	do.	AA	30.1	4 $\frac{1}{2}$ -in. round	0.98	12
211.4	1-1 $\frac{1}{2}$ -3	AA	19.8	4 $\frac{1}{2}$ -in. round	0.98	6
211.5	do.	AA	19.5	do.	0.98	6
331.1	1-2-4	AA	15.7	4 $\frac{1}{2}$ -in. round	0.98	12
331.3	do.	AA	14.9	do.	0.98	12
332.1	do.	AA	15.3	do.	0.98	12
217.0	1-2-4	AA	15.3	4 $\frac{1}{2}$ -in. round	0.98	6
217.1	do.	AA	14.9	do.	0.98	6
333.1	do.	AA	14.5	do.	0.98	12
217.3	1-2-4	AA	16.5	4 $\frac{1}{2}$ -in. round	0.98	6
217.5	do.	AA	14.3	do.	0.98	6
217.2	1-2-4	AA	14.9	4 $\frac{1}{2}$ -in. round	0.98	6
217.4	do.	AA	14.4	do.	0.98	6
218.1	do.	AA	14.7	do.	0.98	6
218.2	do.	AA	14.7	do.	0.98	6
218.3	do.	AA	15.1	do.	0.98	6
218.4	do.	AA	15.1	do.	0.98	6
254.1	do.	AA	14.3	5 $\frac{1}{2}$ -in. round	1.23	8
254.2	do.	AA	15.3	do.	1.23	8
252.5	1-2-4	U	15.0	4 $\frac{1}{2}$ -in. cor. h. s.	1.25	6
252.6	do.	AA	14.7	do.	1.25	6
252.7	do.	U	14.6	do.	1.25	6
253.5	do.	U	14.5	do.	1.25	12
253.6	do.	U	14.6	do.	1.25	6
219.1	1-2-4	AA	14.7	6 $\frac{1}{2}$ -in. round	1.47	6
219.2	do.	AA	14.7	do.	1.47	6
240.2	do.	AA	14.2	3 $\frac{3}{4}$ -in. round*	1.65	6
255.1	do.	AA	15.3	3 $\frac{3}{4}$ -in. round	1.65	10
255.2	do.	AA	13.4	7 $\frac{1}{2}$ -in. round	1.71	10
220.5	do.	U	14.1	2 1-in. round	1.96	6
220.1	do.	AA	16.5	4 $\frac{3}{4}$ -in. round	2.21	6
220.2	do.	AA	14.1	do.	2.21	6
256.1	do.	AA	14.2	do.	2.21	12
256.2	do.	AA	14.6	do.	2.21	12
253.1	do.	AA	14.7	4 $\frac{3}{4}$ -in. cor. h. s.	2.81	12
336.3	do.	AA	14.5	5 $\frac{1}{2}$ -in. round	1.23	12
213.3	1-3-6	AA	11.0	4 $\frac{1}{2}$ -in. round	0.98	6
213.4	do.	AA	9.5	do.	0.98	6
351.2	1-4-8	AA	7.3	dc.	0.98	12

*Washers held by nuts on both faces.

[Continued on page 38.]

TABLE 9.—*Continued.*

TESTS OF BEAMS WITHOUT WEB REINFORCEMENT.

Series of 1908.

All beams failed by diagonal tension unless otherwise noted.

Beam No.	Age days	Load at First Diagonal Crack pounds	Maximum Applied Load pounds	Stress in Longitudinal Reinforcement lb. per sq. in.	Vertical Shearing Stress lb. per sq. in.	Bond Stress lb. per sq. in.	Control Beam	
							Modulus of Rupture lb. per sq. in.	Age days
210.1	63	25 000	25 640	45 800	190	257	492	61
210.3	60	20 000	24 500	43 800	182	246	390	60
210.2*	72	18 000	31 000	55 300	229	309	553	69
343.2	73	12 000	16 900	41 700	134	121	766	69
341.3*	253	7 600	12 000	45 000	96	130	607	251
211.4	63	16 000	18 800	34 100	142	192	395	68
211.5	72	15 900	24 200	43 700	181	245	448	71
331.1	7	7 000	7 750	33 000	72	97	29	3
331.3	4	2 900	2 900	14 400	33	45	140	11
332.1	7	8 000	8 600	36 300	78	105	213	7
217.0	14	15 400	16 200	30 900	128	173	400	63
217.1	15	13 500	14 000	26 800	111	150	227	15
333.1	17	7 000	7 500	31 000	66	89	157	17
217.3	32	17 900	18 500	34 300	143	193		
217.5	37	13 000	13 600	25 400	107	145	278	75
217.2	63		20 000	36 800	152	205	301	67
217.4*	81	16 000	24 700	45 100	187	252	439	87
218.1*	65	19 000	24 900	45 600	188	254	333	70
218.2	67	19 000	23 000	42 200	175	236	422	61
218.3	60	14 000	16 600	30 600	128	178	445	69
218.4	61	17 000	17 300	31 900	133	180		
254.1	63	16 000	16 740	33 400	132	143		
254.2	75	16 000	22 200	44 000	172	166		
252.5	66	25 000	26 000	37 800	199	199		
252.6	66	20 800	20 800	30 400	161	161	452	72
252.7	64	19 000	23 790	34 500	182	182		
253.5	66	17 000	18 450	55 200	147	147	365	59
253.6	64		24 500	35 700	188	188		
219.1	65	25 000	25 200	31 600	193	174	333	70
219.2	67	20 000	21 700	27 200	169	152	422	61
240.2	64	18 000	20 500	23 100	161	182	321	66
255.1	63	16 000	18 900	36 000	151	171	492	64
255.2	63	17 000	20 000	36 700	162	125		
220.5	67	17 000	19 400	18 500	155	98	243	67
220.1	68	23 850	23 850	20 500	190	162		
220.2	66	27 000	27 000	23 000	214	182	427	72
256.1	64	13 000	18 210	31 900	150	128	492	64
256.2	71	18 000	21 000	36 700	172	146	281	64
253.1	67	20 000	21 300	29 700	174	144		
336.3	251	12 000	14 800	45 300	109	111	348	251
213.3	68	20 000	22 150	40 900	170	230	281	68
213.4	67	9 000	13 300	24 900	104	140	197	66
351.2	14		3 340	7 200	32	43	97	14

* Tension failure.

[Continued on page 39.]

TABLE 8.—*Continued.*

DATA OF BEAMS WITHOUT WEB REINFORCEMENT.

Series of 1908.

Beam No.	Kind of Concrete	Cement		Reinforcement		Span Length feet
		Kind	Per cent	Description	Per cent	
214.3	1-4-8	AA	7.3	4 $\frac{1}{2}$ -in. round	0.98	6
214.4	do.	AA	7.7	do.	0.98	6
214.5	do.	AA	7.7	do.	0.98	6
352.1	do.	AA	7.4	do.	0.98	12
352.2	do.	AA	7.5	do.	0.98	12
353.2	do.	AA	7.3	do.	0.98	12
351.1	do.	AA	7.2	do.	0.98	12
215.3	1-5-10	AA	6.5	do.	0.98	6
215.4	do.	AA	6.2	do.	0.98	6
215.5	do.	AA	5.7	do.	0.98	6
361.1	do.	AA	6.8	do.	0.98	12
362.1	do.	AA	6.2	do.	0.98	12
362.2	do.	AA	6.1	do.	0.98	12
363.2	do.	AA	6.1	do.	0.98	12
361.2	do.	AA	6.1	do.	0.98	12
363.1	do.	AA	6.0	do.	0.98	12

what greater area of bar to resist bond stress than is taken into account in the formula. The amount of this is not large and the excess of bond area is less than is generally found in actual reinforced concrete construction; the effect upon calculations of diagonal tension and shear is slight. It should be borne in mind also that in these tests the load was applied by increasing it continuously to failure. It must be expected that repeated applications of a load will have considerable effect upon the resistance of the web to diagonal tension, especially after diagonal cracks have formed. Just what proportion of the load which produces failure in the ordinary test would finally cause failure by repetitive loading is not known. The critical load is probably between that for first crack and the ordinary breaking load.

For reasons already given, the vertical shearing unit-stress will be used as the measure or means of comparison for the resistance of concrete to diagonal tensile stresses, and the values of the vertical shearing stresses are given for this purpose. The test beams were planned to give information along several lines,—(a) the effect of the amount of cement, (b) the effect of the age of the

TABLE 9.—*Continued.*

TESTS OF BEAMS WITHOUT WEB REINFORCEMENT.

Series of 1908.

All beams failed by diagonal tension unless otherwise noted.

Beam No.	Age days	Load at First Diagonal Crack pounds	Maximum Applied Load pounds	Stress in Longitudinal Reinforcement lb. per sq. in.	Vertical Shearing Stress lb. per sq. in.	Bond Stress lb. per sq. in.	Control Beam	
							Modulus of Rupture lb. per sq. in.	Age days
214.3	67	14 450	14 450	27 200	113	153	183	67
214.4	72	12 000	13 200	24 800	103	139	140	70
214.5	64	12 000	13 800	26 000	108	146		
352.1	62	7 500	9 100	36 300	78	105	224	62
352.2	65	7 000	7 400	30 100	65	88		
353.2	73	6 600	8 020	32 400	70	95	160	69
351.1	192	7 000	7 000	28 700	62	84	184	192
215.3	66	9 700	9 700	18 800	80	108	132	79
215.4	71	9 700	10 000	19 300	82	111	176	60
215.5	67	8 900	8 900	17 300	74	93	176	60
361.1	67	6 800	6 800	28 400	62	84	173	70
362.1	67	6 700	6 700	28 000	61	82		
362.2	68	4 300	4 300	19 200	43	58	88	70
363.2	73	3 500	3 500	16 200	37	50	94	69
361.2	254	4 600	4 600	19 600	45	61	38	251
363.1	190	8 010	8 010	32 800	72	97	96	62

test beam, (c) the effect of the length of beam as compared with its depth, and (d) the effect of the amount of reinforcement. These topics will be discussed in order, together with the relation of the strength of the beam to that of the auxiliary test pieces and the amount of the load at which the first diagonal cracks were noted. Generally speaking, the beams were planned to give diagonal tension failure. The results of the tests of several beams made for other purposes have been utilized, however, in order to extend the range of the work, even though the failure was not by diagonal tension. Failures other than by diagonal tension are noted in the tables.

21. *Effect of Amount of Cement.*—The purpose of one set of tests was to find the effect upon web resistance of quality of concrete as influenced by amount of cement used. It is evident, of course, that the richness and the strength of the concrete will have a very decided influence on resistance to web stresses. In

Table 9 (p. 35, 37 and 39) are given the values of the vertical shearing stresses for beams having a variety of proportions of cement. The effect of amount of cement upon resistance to diagonal tension in beams of 6-ft. span at an age of about 60 days is brought out in Fig. 8 in which values for 1 % beams of 6-ft. span tested at an age of about 60 days are platted. The general average values of the vertical shearing unit-stress for beams of 6-ft. span about 60 days old for the two years may be stated as follows: 1-1-2 concrete, 180 and 200 lb. per sq. in.; 1-2-4 concrete, 130 and 150 lb. per sq. in., 1-3-6 concrete, 95 and 120 lb. per sq. in.; 1-4-8 concrete, 80 and 110 lb. per sq. in. The results of a wider range of experiments, reduced to values for 1 % reinforcement and 8½-ft. span length (span = 10 d) by the method which is explained on page 45, are platted in Fig. 12, and are discussed on page 46.

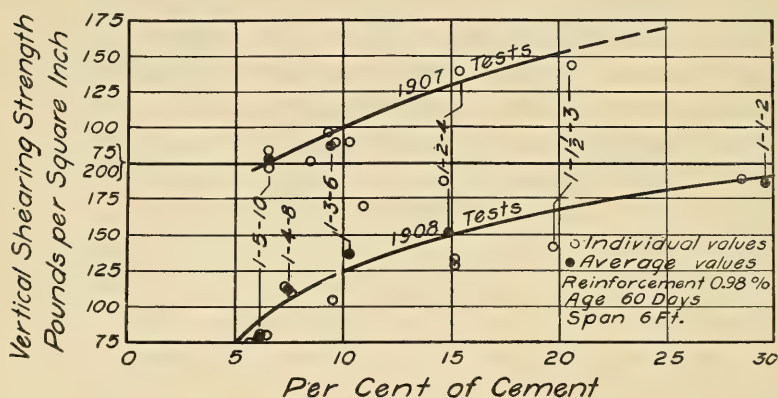


FIG. 8. DIAGRAM SHOWING EFFECT OF AMOUNT OF CEMENT UPON WEB RESISTANCE.

The variation from average values found in individual beams should be noted, and the possibility of such variation suggests the desirability of choosing a lower working stress than would otherwise be necessary. It is evident from these tests that the addition of cement gives a considerable gain in resistance to diagonal tension and that a rich concrete should be used in the web of reinforced concrete beams which are subjected to any considerable amount of diagonal tension, especially if there is no metallic web reinforcement or the web reinforcement is not effective.

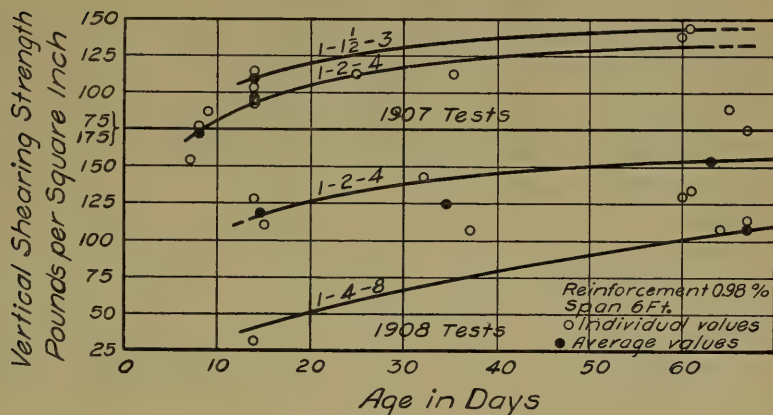


FIG. 9. DIAGRAM SHOWING EFFECT OF AGE UPON WEB RESISTANCE.

22. *Effect of Age.*—It is important to know at what age concrete may be permitted to resist web stresses, as, for example, to know the effect of removal of forms and supports. In Fig. 9 the vertical shearing stress which beams 6-ft. long developed at various ages, taken from Table 9, are platted. These may be expected to represent the web strength of beams of the dimensions used for the temperatures and conditions of the laboratory. The results show considerable variation, as is to be expected. A rapid increase with age may be noted at the earlier ages, and a slower change for the older beams. The lean concretes attain their strength more slowly. Attention is called to the results of Beams No. 336.3, 361.2 and 363.1 in Table 9. These results indicate little or no gain beyond that attained in the ordinary test period.

23. *Relation to Strength of Auxiliary Test Pieces.*—In building construction some auxiliary test may be used with advantage to check up the quality of the concrete placed in beams in which diagonal tension is the critical stress. The writer has suggested the form of flexure test piece here called the "control beam" for this purpose. The modulus of rupture determined from the test of the plain concrete control beam is representative of the tensile strength of the concrete and may be used as its measure, although it does not give the actual tensile strength of the concrete. The resistance of the beam to diagonal tension is dependent upon the tensile strength of the concrete. If the ratio which may be

TABLE 10
VERTICAL SHEARING STRESS, MODULUS OF RUPTURE,
AND COMPRESSIVE STRENGTH.

The ratios given are the ratios of the vertical shearing stress to the modulus of rupture of control beams in one case, and to compressive strength of 6 in. cubes in the other. Beams have various percentages of reinforcement. Except as otherwise noted, the age of all beams, control beams and cubes was approximately 60 days. The exact age and amount of reinforcement are given in Tables 6, 8 and 9.

No. of Beam	Year	Kind of Concrete	Vertical Shearing Stress lb. per sq. in.	Control Beam Modulus of Rupture lb. per sq. in.	Ratio	Crushing Strength of 6-in. Cubes lb. per sq. in.	Ratio
210.1	1908	1-1-2	190	492	.39	3960	.048
210.3	"	"	182	390	.47	3570	.051
					Av. .43		.050
211.3	1907	1-1-3	144	354	.41		
211.1*	"	"	222	331	.67	4590	.048
211.2	"	"	142	310	.46		
211.4	1908	"	142	395	.36	3320	.043
211.5	"	"	181	448	.40		
					Av. .46		.046
212.7	1907	1-2-4	138	336	.41		
212.1	"	"	155	228	.68		
212.2	"	"	162	305	.53		
212.5*	"	"	121	221	.55	3290	.037
212.6	"	"	132	246	.54		
613.2*	"	"	144	280	.51		
616.2*	"	"	190	260	.73		
251.1	"	"	149	291	.51		
251.2	"	"	114	183	.62		
251.3	"	"	102	238	.43		
251.4	"	"	110	288	.38		
417.5	"	"	100	229	.44		
417.6*	"	"	89	189	.47	2420	.037
611.1	"	"	149	293	.51		
611.2	"	"	175	252	.69		
612.1	"	"	153	339	.45		
613.1	"	"	111	307	.36		
616.1	"	"	156	249	.63		
617.1	"	"	169	304	.56		
617.2	"	"	173	304	.57		
217.2	1908	"	152	301	.51	2170	.070
217.4*	"	"	187	439	.43	2910	.064
218.1	"	"	188	333	.56		
218.2	"	"	175	422	.41	2670	.066
218.3	"	"	128	445	.29	2520	.051
218.4	"	"	133			1855	.072
254.1	"	"	132			2100	.063
252.6	"	"	161	452	.36	2330	.069
252.7	"	"	182			3055	.060
253.5	"	"	147	365	.41	3660	.040

* For the age of these tests see Tables 6, 8 and 9.

TABLE 10.—*Continued.*

No. of Beam	Year	Kind of Concrete	Vertical Shearing Stress lb. per sq. in.	Control Beams Modulus of Rupture lb. per sq. in.	Ratio	Crushing Strength of 6-in. Cubes lb. per sq. in.	Ratio
253.6	1908	1-2-4	188			3060	.061
219.1	"	"	193	333	.58		
219.2	"	"	169	422	.40	2670	.063
240.2	"	"	161	321	.50	1780	.090
255.1	"	"	151	492	.31	2230	.068
255.2	"	"	162			2460	.066
220.5	"	"	155	243	.64	2210	.070
220.2	"	"	214	427	.50	2330	.092
256.1	"	"	150	492	.31	2230	.067
256.2	"	"	172	281	.61	2460	.070
253.1	"	"	174			2330	.075
					Av. .50		.064
213.1	1907	1-3-6	98	178	.55		
213.2	"	"	90	204	.44		
213.3	1908	"	170	281	.61	1690	.101
213.4	"	"	104	197	.53	1120	.093
					Av. .53		.097
214.1*	1907	1-4-8	90	179	.50	1315	.068
214.2	"	"	77	174	.44		
415.1	"	"	75	135	.56		
415.2	"	"	64	188	.34		
415.5	"	"	62	151	.41		
415.6	"	"	70	125	.56		
214.3	1908	"	113	183	.62		
214.4	"	"	103	140	.74	1305	.079
352.1	"	"	78	224	.35	1570	.050
352.2	"	"	65			1010	.064
353.2	"	"	70	160	.44	820	.085
					Av. .50		.069
215.1	1907	1-5-10	84	174	.48		
416.5	"	"	49	109	.45		
416.6	"	"	54	118	.46		
215.3*	1908	"	80	132	.61	1100	.073
215.4	"	"	82	176	.47	1175	.070
215.5	"	"	74	176	.42	900	.082
361.1	"	"	62	173	.36	1025	.060
362.1	"	"	61			870	.070
362.2	"	"	43	88	.49	900	.048
363.2	"	"	37	94	.39	600	.062
					Av. .46		.066

* For the age of these tests see Tables 6, 8 and 9.

expected between the vertical shearing strength of the reinforced concrete beam and the modulus of rupture of the control beam be established, this ratio and the flexural strength of the control beam may be used as a measure of the quality of the concrete to resist web stresses. It will be seen that this ratio may vary with different proportional dimensions of beams and with different amounts of cement, but if the ratios be determined for the various conditions, this method should be helpful as an aid to securing substantial construction.

Table 10 gives the ratios of the vertical shearing unit-stress of the reinforced concrete beams to the modulus of rupture of the control beam for the test beams for which control beams were made in those cases where the age of both test beam and control beam was 60 days or thereabouts, and for a few other cases. As only one control beam was made for each test beam, it may be expected that there will be considerable variation in the results. The average of the ratios is about 0.50.

The values of the modulus of rupture of the control beams may be helpful, in connection with other tests, in determining what values should be specified to secure a desired grade of concrete.

The ratio of the vertical shearing unit-stress to the compressive strength given by the cube tests is also given in Table 10 for the cases where the conditions of test are comparable. The ratios are fairly uniform. The usefulness of making field test pieces for tests such as the flexure test and the compression test of cubes or cylinders is a subject worthy of consideration by constructors.

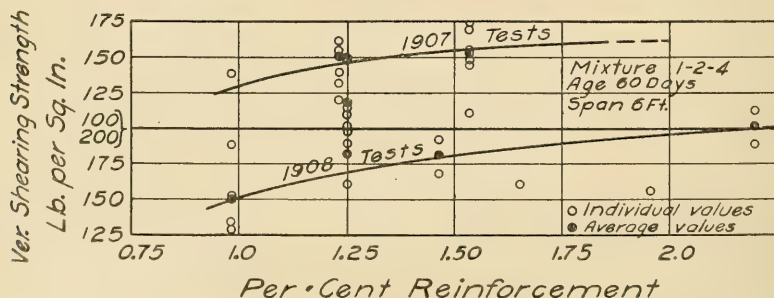


FIG. 10. DIAGRAM SHOWING EFFECT OF AMOUNT OF REINFORCEMENT UPON WEB RESISTANCE

24. *Effect of Amount of Reinforcement.*—Fig. 10 gives values of vertical shearing stress developed with the several percentages of reinforcement used. These beams were all of 6 ft. span and 1-2-4 concrete and were tested at about 60 days' age and all failed by diagonal tension. It is evident that the shearing stress developed in these tests is greater for beams with the larger reinforcement. It is possible that this change is due to the greater deformation (or greater curvature in the elastic curve) of the beams having light reinforcement. Possibly the fact that at the same load tension cracks will be present in the concrete, at points nearer the supports in the beam with the lighter reinforcement on account of the higher tensile stresses developed in the reinforcing steel, has a bearing upon the difference in web resistance. That the bond stresses developed are not the occasion of the variation may be judged from the fact that the beams reinforced with deformed bars carried no higher loads than those with plain round rods.

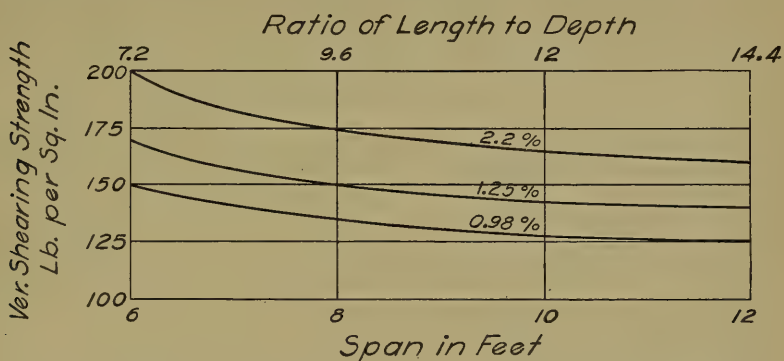


FIG. 11. EFFECT OF SLENDERNESS OF BEAM UPON WEB RESISTANCE.

Fig. 10 has been used for the purpose of reducing the values of shear found in the various tests to the basis of 1% reinforcement, the amount of the stress being plotted at the proper point and then carried to the 1% line by keeping proportionally distant from the curves shown in the figure. This enables further comparisons to be made.

25. *Effect of Ratio of Length of Span to Depth of Beam.*—In Fig. 11 have been plotted values of web resistance for three percentages of reinforcement and for four spans and length ratios, which are thought to be representative of the results of the tests for

1908. These have been selected from average values and reduced to a common basis by the use of the other diagrams. The effect of slenderness of beam, as well as of amount of reinforcement, is brought out in this diagram. The figure has been used in reducing other results to a common basis, in a manner similar to that described in the preceding paragraph.

26. *Appearance of First Diagonal Crack.*—The applied load at which the appearance of a diagonal crack was noted is given in Table 9. In some of the tests no special care was taken to observe

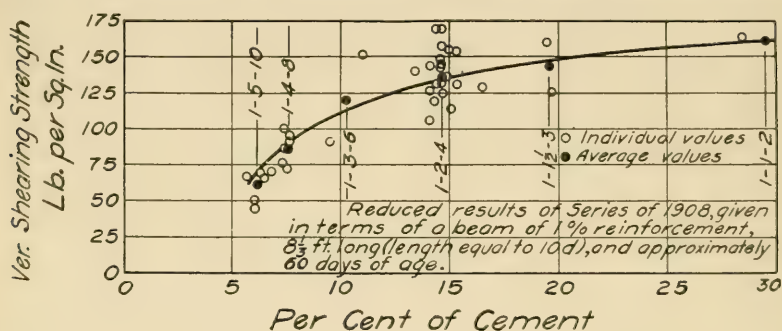


FIG. 12. VALUES FOR WEB RESISTANCE.

the cracks and this may account for part of the variability shown. In the younger and leaner concretes the crack was noted but little before the maximum was reached, and frequently not until failure occurred. In the richer and stronger concretes more warning generally was given, and in some beams which carried high loads the diagonal crack was visible for some time before failure.

27. *Value of Vertical Shearing Stress.*—In Fig. 12 are plotted results of tests for 1908—all reduced to a basis of 1% reinforcement and 8½-ft. span length (10d) by the use of Fig. 11, as previously described. It is seen that there is a great variation in the results, and this may be expected in a material like concrete. In the selection of working stresses, allowance for the possibility of such variations in reinforced concrete construction should be made.

The marked effect of the addition of cement is of interest, and it is evident that an effective way to provide higher web resistance is to increase the richness of the concrete. Attention is also called to the very low strengths found in the 1906 tests, reported in Bulletin No. 14.

Diagonal tension weakness in reinforced concrete construction should be guarded against. In failures by tension in steel or by compression in concrete warning may be expected through abnormal deflections, but in beams without web reinforcement diagonal tension failures occur suddenly and generally without warning. It seems important, therefore, that relatively low working stresses in shear be used or that effective web reinforcement be provided, and that special care be taken in the construction of parts having high shearing stresses.

B. BEAMS WITH REINFORCING BARS BENT UP.

28. *Beams with Reinforcing Bars Bent up*—The beams of this series were of 1-2-4 concrete and 6-ft. span length. The variations included (see Fig. 13, 14, and 15) (a) bars bent up to a point $2\frac{1}{2}$ in. below the top of the beam at the end, (b) bars bent up to 5 in. below the top, (c) bars bent up to $2\frac{1}{2}$ in. below the top and nuts and washers attached at the end of the bars, (d) bars bent up to 5 in. below the top and nuts and washers attached, and (e) part of the bars bent up and part of them straight. There will also be included in this class the beams made with the Cummings' unit frame. This form of reinforcement is described on page 26. Table 11, page 56, and Table 12, page 57, give data of beams and tests.

29. *Phenomena of Tests of Beams with the Reinforcing Bars Bent Up*.—As may be expected, in the beams with all the bars bent up, as soon as the concrete at the bottom was sufficiently stretched, a vertical tension crack formed at some point between the beam support and the bend of the bar, extending from the bottom of the beam to the level of the reinforcing bars, (see Fig. 13, 14, and 15). After the formation of this vertical crack, the tension in this part of the beam is taken mainly by the reinforcing bar; and as its depth below the top of the beam (and therefore the moment arm of the resisting forces) is considerably less than in beams having the bars horizontal throughout, the tensile stress developed in the bar at this section is greater than in beams having the reinforcement horizontal throughout. The vertical shearing unit-stress actually developed is also greater than in beams having the bars horizontal, and the resulting diagonal tensile stress is therefore greater. As the tensile stress in the steel at

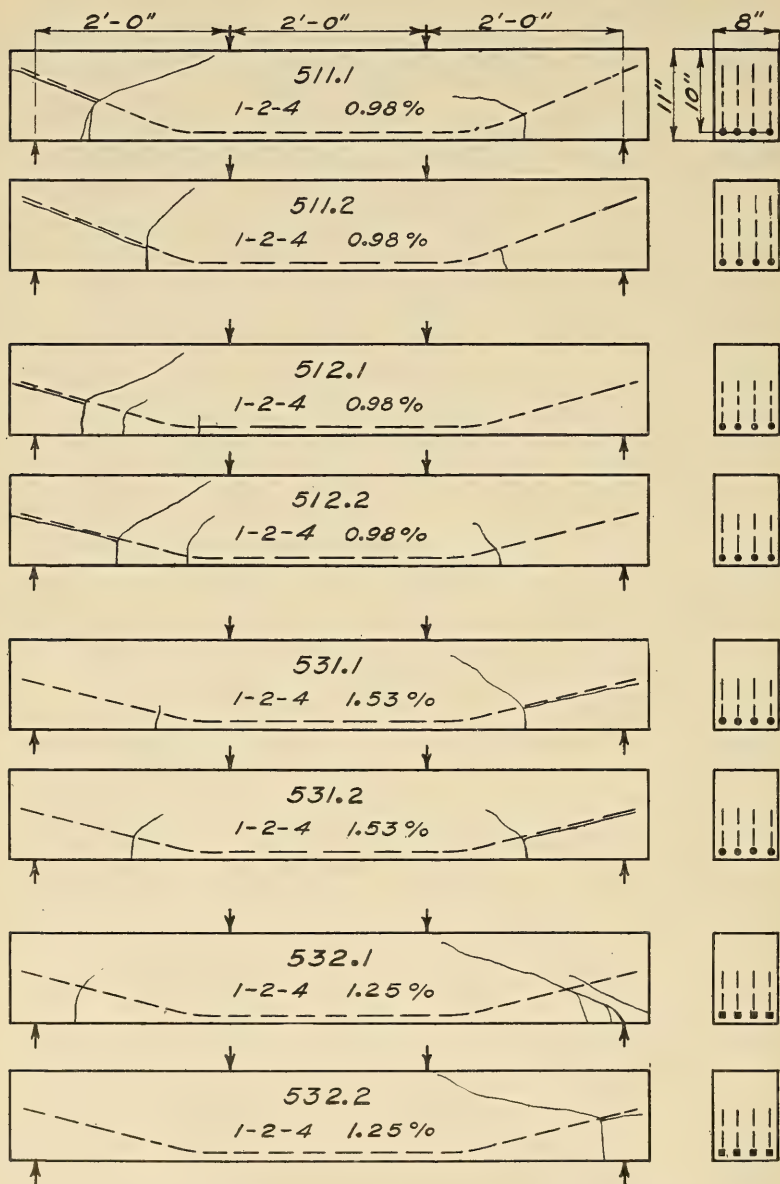


FIG. 13. DISPOSITION OF REINFORCEMENT AND POSITION OF CRACKS.

this section is greater than when the bars are horizontal, the amount of bond brought into action toward the end of the bar must be greatly increased. There is evidently, then, a greater chance for slip of bar in such beams than in beams where straight reinforcement is used. The formation of the vertical cracks gives a good starting point for diagonal tension cracks, and the uneven distribution of stresses in the concrete due to this crack makes diagonal tension failures occur at a lower load than would otherwise be the case. Generally, in these tests, in the beams which did not fail by tension in the steel, the failure came through diagonal tension. The inclined crack which opened up along the line of the bent-up bars, as the concrete below gradually split off, may be said to be the occasion of final failure, though the failure was essentially by diagonal tension. Whether slip occurred after this crack extended along the bars nearly to their ends is a matter of some interest.

The action of the beams having nuts and washers and of beams with part of the bars straight will be seen to be somewhat different from the foregoing.

The following are brief notes of the tests. The location of the cracks is shown in Fig. 13, 14, and 15. The heavy lines indicate cracks along which failure took place. Reference may be made to Tables 11 and 12.

No. 511.1. At 13 000 lb. the first noticeable crack appeared at 2 ft. to right of center and extended from the bottom vertically 3 in. to the level of the reinforcement and then diagonally toward the load point 5 in. At 14 000 lb. a vertical crack appeared 2 ft. 5 in. to left of center and extended to the reinforcing bars. From this point it finally ran diagonally almost to the load point. The beam failed at 18 000 lb., the crack following the reinforcing bars to the end of the beam. The load then dropped quickly to 6000 lb. where it remained constant for some time.

No. 511.2. At 13 000 lb. a crack appeared 1 ft. 10 in. to left of center and extended to the reinforcing bars. At 15 000 lb. this crack had extended diagonally almost to the top of the beam, 4 in. to the left of the load point. The load of 15 000 lb. was held for 15 minutes, due to a break in the machine. The beam failed at 20 000 lb., the load then dropped slowly to 4000 lb., and the crack on the left extended along the reinforcement almost to the end of the beam. An examination of the beam made afterward showed that the bars had slipped $\frac{1}{2}$ in. in the left end.

No. 512.1. At 15 000 lb. two cracks were noted, one 2 ft. 5 in. to the right and the other 2 ft. 1 in. to the left of center, extending upward to the reinforcing bars. At 18 000 lb. a crack appeared 2 ft. 6 in. to the left of center and extended to the reinforcing bars, branching off diagonally to the right toward the load point and to the left along the reinforcing bars. The maximum load was 19 000 lb. and the load gradually dropped to 2000 lb. An examination of the beam showed that the rods at the left end had slipped slightly. The crack had followed the reinforcing bars to the end.

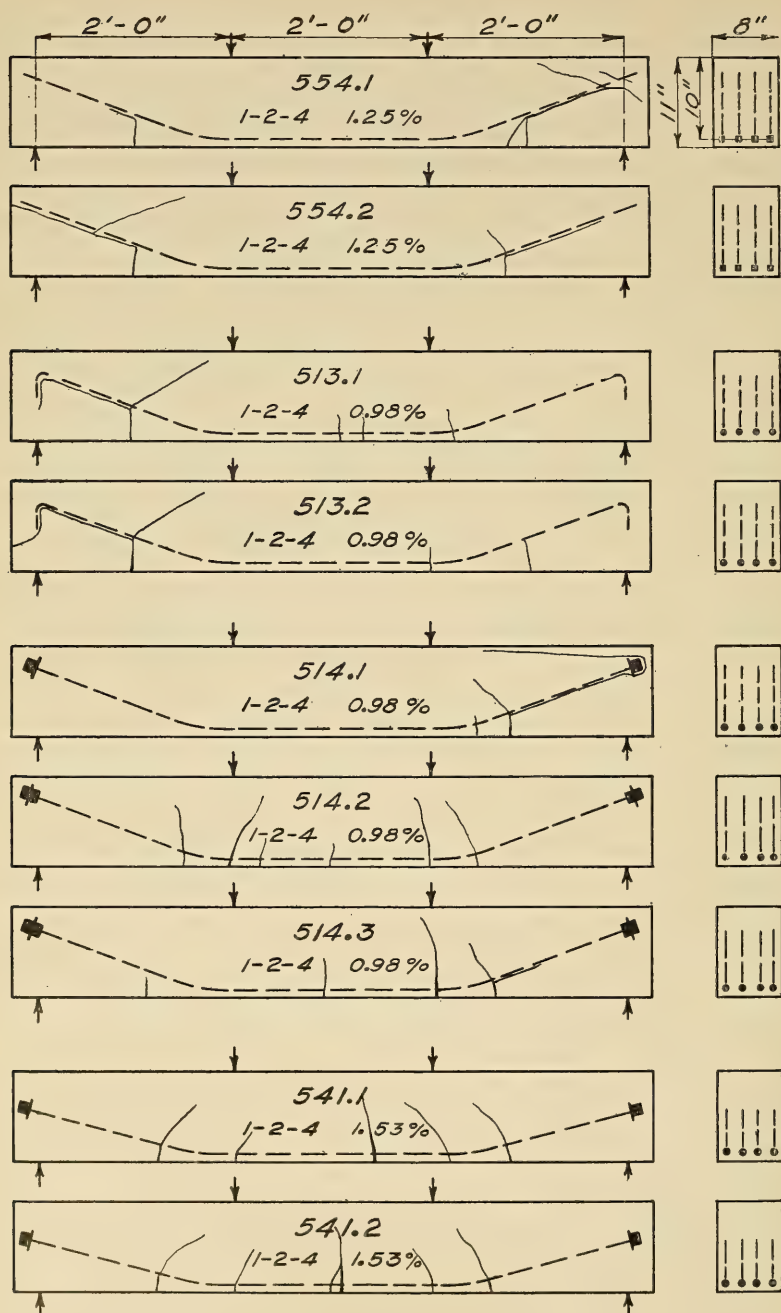


FIG. 14. DISPOSITION OF REINFORCEMENT AND POSITION OF CRACKS.

No. 512.2. At 14 000 lb. a crack appeared 1 ft. 5 in. to left of center and extended toward the load point. At 16 000 lb. a crack was observed 2 ft. 2 in. to left of center, which extended vertically to the reinforcing bars and from here started toward the load point. The beam failed quite suddenly at a load of 17 350 lb., and the crack followed the reinforcing bars to the end of the beam. An examination of the beam showed that the bars had slipped $\frac{1}{8}$ in.

No. 531.1. This beam failed in a manner similar to No. 512.1 and at a load but 500 lb. greater. The crack at 1 ft. 10 in. to right of center appeared at 14 000 lb. and the beam failed at 19 500 lb., the crack following the reinforcing bars to the end of the beam.

No. 531.2. At 12 000 lb. a crack appeared 2 ft. to left of center and extended vertically to the reinforcing bars. At 18 000 lb. this crack was 5 in. above the bottom and was extending toward the left load point. At 14 000 lb. a similar crack appeared 2 ft. to right of center and extended in the same way. At 22 000 lb. a slight hair crack was noted following the line of the reinforcement. The beam failed by the opening of the crack at 24 000 lb.

No. 532.1. At 18 000 lb. a diagonal crack appeared at 2 ft. 5 in. to right of center and the beam finally failed along this crack. The maximum load was 20 000 lb.

No. 532.2. At the maximum load of 18 000 lb. a diagonal crack extended from the right end toward the load point. The bars slipped after the diagonal tension failure occurred.

No. 554.1. At 13 600 lb. a vertical crack appeared 1 ft. 10 in. to left of center and one 2 ft. to the right of center which extended 4 in. high to another crack. At 17 900 lb. the beam failed as shown in the sketch in Fig. 14.

No. 554.2. At 10 800 lb. a vertical crack appeared 2 ft. to the left of center, extending up to the reinforcement, thence along the reinforcement to the end of the beam. A similar crack appeared 1 ft. 9 in. to the right of center. At a maximum load of 17 500 lb. the beam failed along the crack to the left of center.

No. 513.1. At 9 000 lb. a crack appeared 2 ft. to left of center and extended vertically to the reinforcing bars. The beam failed suddenly at 16 700 lb. The crack followed the reinforcement to its end.

No. 513.2. At 10 000 lb. a crack appeared 2 ft. to left of center, extending vertically to the reinforcing bars. As the load increased this crack extended diagonally toward the load point. The beam failed at 16 400 lb.; this crack following the reinforcing bars to the end.

No. 514.1. At 10 000 lb. a crack appeared at 1 ft. 10 in. to right of center and extended to the reinforcement. The beam failed suddenly at 21 100 lb. by this crack running along the reinforcement to the nuts. At failure a crack appeared at the nut, running back toward the load point. The diagonal crack shown closed up. This is a peculiar failure and differs from others of its set. As noted elsewhere the washers were not locked to the end nuts in this beam.

No. 514.2. This beam is noteworthy because it failed through tension in the steel, although a crack was visible along the reinforcing bars. At 14 000 lb. a crack appeared to the right of the right load point and finally extended some distance toward the load point. At 19 000 lb. slight cracks were seen following the reinforcement from this crack. At 16 000 lb. a vertical crack 3 in. long appeared at the center and also a crack below the left load point which extended upward 4 in. At the maximum load of 22 200 lb. this crack was 8 in. high, and opening considerably, showing failure by tension in the steel. At 18 000 lb. a vertical crack 6 in. long was noted 1 ft. 6 in. to left of center and at the maximum load it was 8 in. long. The deflection of the beam at the maximum load was 0.2 in. and the load remained within 1000 lb. of the maximum until the deflection was 0.43 in. Crushing of the concrete at the top of the beam occurred under the increased deflection.

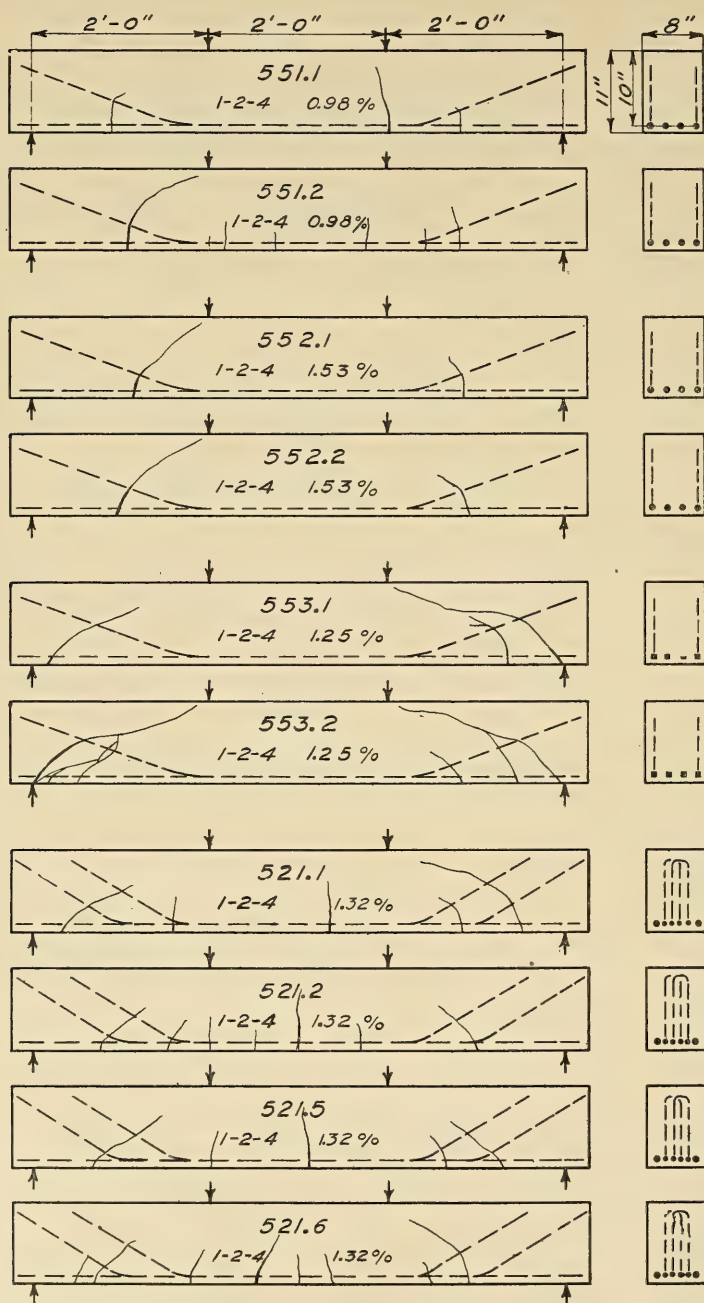


FIG. 15. DISPOSITION OF REINFORCEMENT AND POSITION OF CRACKS.

No. 514.3. At 12 000 lb. a crack appeared 1 ft. 8 in. to right of center and extended vertically 4 in. At 18 000 lb. a crack had branched off from the first crack and followed the reinforcement for a short distance. At 22 000 lb. a crack appeared under the right load. At the maximum load of 23 500 lb. this crack was 8 in. high and as the deflection of the beam was increased the crack opened up almost to the top of the beam and finally the concrete began to crush at the top. The beam failed by tension in the steel. The effect of the anchorage is shown in that the inclined crack did not extend to the end of the bar.

No. 541.1. At 16 000 lb. the first crack appeared at 1 ft. 10 in. to left of center and was $2\frac{1}{2}$ in. long. At 18 000 lb. it had extended toward the load point until it was 5 in. from the bottom of the beam and at 28 000 lb. it was 7 in. A similar crack appeared at 1 ft. 10 in. to right of center at 18 000 lb. and at 26 000 lb. this crack was visible 7 in. above the bottom of the beam. At 18 000 lb. cracks were noted under the two load points. At 22 000 lb. a crack 5 in. to right of center was 2 in. high. At the maximum load of 29 600 lb. this crack was 6 in. long. This load was held almost constant for some time and the beam failed by tension in the steel, followed by crushing of the concrete at the top of the beam.

No. 541.2. This beam failed in a manner similar to No. 541.1. At 12 000 lb. the first crack appeared 1 ft. 7 in. to right of center and extended 4 in. toward the load point. At 14 000 lb. a similar crack appeared 1 ft. 10 in. to left of center. At 16 000 lb. this crack was 8 in. above the bottom of the beam. At 16 000 lb. a small vertical crack was observed near the center. The maximum load was 28 450 lb. The beam failed by tension in the steel.

No. 551.1. At 14 000 lb. a crack appeared 1 ft. 10 in. to right of center, extending vertically 3 in. At 16 000 lb. a similar crack appeared 2 ft. 1 in. to left of center. At 22 000 lb. a slight vertical crack was noted under the right load point. This gradually extended upward until the maximum load of 24 700 lb. was reached. The beam failed by tension in the steel and an examination showed that the rods had scaled. The cracks in the other one-third did not develop to any extent.

No. 551.2. At 14 000 lb. two cracks appeared: one 2 in. long at the right one-third point and one 4 in. long near the left one-third point. At 16 000 lb. a vertical crack 6 in. long was noted 1 ft. 11 in. to left of center. At 18 000 lb. three more vertical cracks were noted in the middle third. The beam failed at a load of 22 000 lb. by the crack in the left third of the beam extending to the load point and at the same time following the line of horizontal reinforcement. This diagonal tension failure was at a load which nearly developed the elastic limit of the steel.

No. 552.1. At 22 000 lb. a crack appeared 10 in. to left of left load point 5 in. high and at 23 000 lb. one 14 in. to right of right load point 4 in. high. Failure at 25 000 lb. by the extension of the first crack to the load point.

No. 552.2. At 24 000 lb. a crack appeared 10 in. to right of right load point 5 in. high, and at 25 000 lb. one 17 in. to right of right load point 5 in. high. At 28 000 lb. these cracks were opening and lengthening. At 30 000 lb. the cracks were extended well toward the load point. Final failure came at the left end, the crack reaching from support to load point.

No. 553.1. At 24 000 lb. a diagonal crack appeared 2 ft. 4 in. to right of center and at 24 700 lb. one at 2 ft. 10 in. to left of center. These cracks gradually lengthened and at 28 800 lb. the beam failed by diagonal tension. It will be noted that the failure crack extended to the support.

No. 553.2. At 24 000 lb. a diagonal crack appeared 1 ft. 10 in. to right of center, at 25 000 lb. another at 2 ft. 5 in. to right of center, and at 26 000 lb. one at 2 ft. 6 in. to left of center. These cracks gradually opened and lengthened and at 31 000 lb. the maximum load was reached. The crack at

the right end extended almost to the load point, and the final failure came at the crack at the left end which reached from support to load point.

No. 521.1. This is the Cummings' unit frame reinforcement. It should be noted that the cracks outside the load points were small and did not extend far, and that failure was by tension in the steel at a high vertical shearing stress. At 20 000 lb. the first crack appeared at 4 in. to right of center and extended upward 4 in. At 22 000 lb. a crack appeared 2 ft. 6 in. to right of center, extending upward 6 in. Another crack 4 in. long was noted 1 ft. 5 in. to the left of center, inclined toward the load point. At 28 000 lb. slight vertical cracks were noted 1 ft. 10 in. to right of center and 2 ft. 8 in. to left of center. The beam failed by tension in the steel at the crack near the center, the maximum load being 31 100 lb. The load ran near the maximum for some time as the deflection was increased and finally the concrete crushed slowly at the top.

No. 521.2. Another Cummings' reinforcement. The first noticeable crack was at the center of the beam at 16 000 lb., extending upward 3 in. At 18 000 lb. a slight crack appeared at the left one-third point. At 20 000 lb. two cracks appeared both inclined toward the load points, one 2 ft. 3 in. to left of center and the other 2 ft. to right of center. At 22 000 lb. a vertical crack appeared at 8 in. to right of center and at 24 000 lb. at 6 in. to left. At 26 000 lb. a crack 6 in. long appeared 1 ft. 6 in. to left of center and inclined toward the load point. At the maximum load of 35 000 lb. the crack at the center had gradually extended, opening up, and the beam failed by tension in the steel. The load remained near the maximum under further deflection and crushing of the concrete at the top of the beam finally occurred. The beam held 34 000 lb. until a deflection of nearly 0.5 in. had been reached. The outer cracks had not developed particularly and there was no appearance of cracks in the direction of the reinforcement.

No. 521.5. Another Cummings' reinforcement. At 20 000 lb. a crack appeared 2 ft. 3 in. to right of center, inclined slightly toward the left. At 24 000 lb. slight vertical cracks were noted at the left one-third point. At 26 000 lb. a crack appeared 2 ft. 4 in. to left of center and inclined toward the load point. At this load a vertical crack 2 in. long was observed 1½ in. to right of center. At 32 500 lb. this crack had opened almost to the top of the beam and the beam failed by tension in the steel. The load carried dropped and crushing of the concrete finally occurred.

No. 521.6. Another Cummings' reinforcement. At 16 000 lb. an inclined crack 6 in. long was visible 2 ft. 4 in. to left of center. At 18 000 lb. an inclined crack 1 ft. 11 in. to right of center appeared which at 26 000 lb. had extended 8 in. from the bottom of the beam. It finally reached almost to the load point. At 18 000 lb. a vertical crack 2 in. long appeared 6 in. to left of center. At 20 000 lb. a vertical crack 2 in. long was noted at the center of the beam. The beam carried a maximum load of 26 200 lb. The deflection diagram is somewhat different from the other beams. The method of failure was evidently by tension in the steel aided by a yielding of the concrete which decreased the moment arm. The stress in the steel is therefore greater than the calculations make it. There was evidence that the concrete in this beam was not so good as in the other beams.

30. *Web Resistance with All Bars Bent Up.*—It seems to make little difference in results, either as to the load carried or as to the manner of failure, whether the bars were bent up to a point close to the top of the beam or to a point 5 in. below the top. For No. 511.1 and 511.2 the value of the nominal vertical shearing stress developed was 145 lb. per sq. in. and for No. 512.1 and 512.2, 140

lb. per sq. in. In both sets vertical cracks (see Fig. 13, 14, and 15) appeared in the outer thirds of the span length, running up to the reinforcing bars. These cracks formed at a nominal vertical shearing stress averaging 105 lb. per sq. in. in No. 511.1 and 511.2, and 112 lb. per sq. in. in No. 512.1 and 512.2. These cracks generally appeared at points about 7 in. from the supports, though in one beam the crack which caused failure was 14 in. from the support. The diagonal crack extended well to the load point before the maximum load was reached. Under these conditions the inclined crack along the reinforcing bars extended and opened, due mostly to the stiffness of the lower segment of the beam next to the support which had no stresses tending to bend it. At final failure there was a slip of the bars at their ends, although the diagonal crack was the real cause of failure. The opening or extension of the inclined cracks helped to concentrate the bond stresses near the end of the bar until these stresses must have become very large. The result of No. 513.1 and 513.2, in which the bars were hooked downward at their ends, indicates that the bars were peeled off, so to speak, although in this case the tension in the rod would have to be carried around its end through the concrete. In passing, it may be noted that it had been planned to hook these bars upward but the workmen made the bend downward.

No. 531.1 and 531.2, which had a higher percentage of reinforcement and therefore had developed less stretch in the steel and less bond stress at a given load and less deflection, gave results higher than 512.1 and 512.2, say 20 per cent more, though the method of failure was the same. The amount of bond surface available was 20 per cent greater. No. 532.1 and 532.2, which were reinforced with corrugated bars, developed a value of the nominal vertical shearing stress of 150 lb. per sq. in.; No. 554.1 and 554.2 gave lower results, 138 lb. per sq. in. As the bond surface available in these four beams was about the same as in No. 531.1 and 531.2 and as the deformed bar would have much greater bond resistance, it would seem that the failure of the beams with bars bent up was not due primarily to slip of bars. A more probable occasion of failure is the splitting of the concrete along the line of the inclined bars after the diagonal cracks have formed by reason of the lack of flexural action in the concrete below the inclined

TABLE 11.

DATA OF BEAMS WITH REINFORCING BARS BENT UP.

Series of 1907.

Span length 6 feet.

Beam No.	Kind of Concrete	Cement		Longitudinal Reinforcement		
		Kind	Per cent	Description	Per cent	Disposition
511.1	1-2-4	AA	16.7	4 $\frac{1}{2}$ -in. round	0.98	Bent to 2 $\frac{1}{2}$ in. of top
511.2	do.	AA	16.5	do.	0.98	do.
512.1	do.	AA	16.3	do.	0.98	Bent to 5 in. of top
512.2	do.	AA	15.7	do.	0.98	do.
531.1	do.	AA	15.7	4 $\frac{5}{8}$ -in. round	1.53	do.
531.2	do.	AA	15.5	do.	1.53	do.
532.1	do.	AA	14.6	4 $\frac{1}{2}$ -in. cor. h. s.	1.25	do.
532.2	do.	AA	14.3	do.	1.25	do.
554.1	do.	AA	17.5	do.	1.25	Bent to 2 $\frac{1}{2}$ in. of top
554.2	do.	AA	14.3	do.	1.25	do.
513.1	do.	AA	15.1	4 $\frac{1}{2}$ -in. round	0.98	Bent to 2 $\frac{1}{2}$ in. of top and
513.2	do.	AA	15.6	do.	0.98	hooked downward at ends
514.1	do.	AA	16.0	do.	0.98	Bent to 2 $\frac{1}{2}$ in. of top, with
514.2	do.	AA	15.8	do.	0.98	nuts and washers
514.3	do.	AA	15.5	do.	0.98	do.
541.1	do.	AA	15.9	4 $\frac{5}{8}$ -in. round	1.53	Bent to 5 in. of top, with
541.2	do.	AA	16.0	do.	1.53	nuts and washers
551.1	do.	AA	15.6	4 $\frac{1}{2}$ -in. round	0.98	Two bars to 2 $\frac{1}{2}$ in. of top,
551.2	do.	AA	15.5	do.	0.98	two bars straight
552.1	do.	AA	15.1	4 $\frac{5}{8}$ -in. round	1.53	do.
552.2	do.	AA	15.9	do.	1.53	do.
553.1	do.	AA	15.9	4 $\frac{1}{2}$ -in. cor. h. s.	1.25	do.
553.2	do.	AA	15.1	do.	1.25	do.
521.1	do.	AA	15.8	Cummings' welded loops, 2 $\frac{5}{8}$ -in. and 4 $\frac{5}{8}$ -in. round	1.32	Bent up at two points and fastened to cross ties
521.2	do.	AA	15.9		1.32	
521.5	do.	U	15.2		1.32	
521.6	do.	U	16.1		1.32	

part of the bars. Beams with the heavier reinforcement would have less deflection for the same load and as this gives a flatter elastic curve the tendency for the inclined crack to form would be less.

TABLE 12.

TESTS OF BEAMS WITH REINFORCING BARS BENT UP.

Series of 1907.

All beams failed by diagonal tension unless otherwise noted.

Beam No.	Age days	Load at First Diagonal Crack pounds	Maximum Applied Load pounds	Stress in Longitudinal Reinforcement lb. per sq. in.	Nominal Vertical Shearing Stress lb. per sq. in.	Nominal Bond Stress lb. per sq. in.	Control Beam	
							Modulus of Rupture lb. per sq. in.	Age days
511.1	70	13 000	18 000	33 100	138	186	231	60
511.2	65	13 000	20 000	36 800	153	207	363	64
512.1	70	15 000	19 000	55 000	146	197	241	60
512.2	63	14 000	17 400	32 100	134	181	275	62
531.1	63	14 000	19 500	23 500	153	156	207	62
531.2	62	12 000	24 000	28 900	187	191	346	63
532.1	62	18 000	20 600	30 000	159	159	291	63
532.2	61	18 000	18 000	26 300	140	140	288	61
554.1	61		17 900	26 200	139	139		
554.2	61		17 500	25 600	136	136	238	61
513.1	70		16 700	30 800	129	174	239	60
513.2	62	10 000	16 400	30 300	127	172	284	63
514.1	65	10 000	21 100	38 700	161	218	207	64
514.2*	62	14 000	22 200	40 700	179	228	253	62
514.3*	63		23 500	43 000	179	242	346	63
541.1*	61	16 000	29 600	35 400	229	234	245	60
541.2*	62	12 000	28 450	34 200	220	224	304	61
551.1*	63	14 000	24 700	45 200	188	254	284	63
551.2	67	14 000	22 000	40 400	168	227	294	60
552.1	59	22 000	25 000	30 100	194	198	352	59
552.2	59	20 000	29 200	35 000	226	230	266	60
553.1	59	24 000	28 800	41 700	220	220	266	60
553.2	59	24 000	31 000	45 000	240	240	352	59
521.1*	63	22 000	31 100	42 800	238	...	213	62
521.2*	67	20 000	35 000	48 200	266	...	245	60
521.5*	61	20 000	32 500	44 800	248	...	313	60
521.6*	62	16 000	26 200	36 200	201	...	231	61

* Tension failure.

The values for the nominal vertical shearing stress in the beams under consideration do not differ much from the vertical shearing stresses developed in the beams of the 1907 series having reinforcing bars horizontal, and hence no gain in resistance was obtained by bending up the bars in this way. Of course,

if calculations be made on the basis of the distance of the bars from the top of the beam at the point where the diagonal crack started, the stress so calculated will be two or three times as great. For the same reason the bond stress near the end of the bars will be much higher than the nominal bond stress given in the tables. There is, therefore, a high tendency to slip, particularly after the inclined cracks have opened some distance. There is an advantage over the construction with the bars straight in that failure of the beams was less sudden and there was more warning.

Attention is called to the low strengths found in beams with poor concrete in the 1906 tests (Bulletin No. 14, page 27).

31. *Web Resistance with Anchored Bars.*—Of the beams having the reinforcing bars anchored with nuts and washers, No. 514.1 failed suddenly at a nominal vertical shearing stress of 161 lb. per sq. in., but the segment at the support split off as shown in Fig. 14, and the small diagonal crack which had formed closed up. It should be noted that in this beam the washers were not fastened against the nuts and there was chance for slip before the washers had solid anchorage. As the other two beams of this set contained a second nut which locked the washer in position, it is possible that this difference in construction explains the lower value and the failure of No. 514.1. In No. 514.2 and 514.3 an inclined crack following the bars for a short distance was visible at one end of the beam for some time before failure, but the manner of failure was by tension in the steel. Beams No. 541.1 and 541.2, having 1.53 % reinforcement and nuts and washers at the ends, also failed by tension of the steel, developing the high nominal vertical shearing stresses of 229 and 220 lb. per sq. in. The diagonal cracks which had formed were closer to the load points than were those in beams having the bars anchored. The general distribution and direction of the cracks are quite different from those of the other beams, and it appears that considerable arch action must have developed. It seems evident from these tests that anchorage of bars at the ends, in beams with bars bent up, if securely arranged, may be advantageous in increasing web resistance. It is true, also, that this form of construction is an insurance against failure at low loads through defective concrete or insufficient bond.

32. *Web Resistance with Part of the Bars Bent Up.*—In the beams in which two bars were bent and two were left straight, vertical cracks formed outside the load points at about the same loads as in the other beams. No. 551.1 failed by tension in the steel at a nominal vertical shearing stress of 188 lb. per sq. in. without the diagonal tension cracks extending very far. In No. 551.2 failure by diagonal tension came at a nominal shearing stress of 168 lb. per sq. in., the diagonal cracks extending to the load point and a crack following the line of the horizontal reinforcement. No. 552.1 and 552.2 failed by diagonal tension at an average nominal vertical shearing stress of 210 lb. per sq. in. It will be noted that the bars of these beams have more bond surface available and that the increased amount of reinforcement makes a stiffer beam than No. 551.2. No. 552.1 and 552.2, reinforced with round rods, and No. 553.1 and 553.2, reinforced with corrugated bars, developed high resistance to diagonal tension, the nominal vertical shearing stress averaging 210 and 230 lb. per sq. in., respectively. This is a high web resistance. The position and distribution of the cracks are quite different from those in the preceding beams.

It is seen that beams with part of the bars bent up and part left straight developed higher resistance to diagonal tension than beams with all bars horizontal or beams with all bars bent up. It is evident also that to secure best results in providing against diagonal tension stresses, the diagonal bars should be well distributed through the part of the beam where high web resistance is needed. This may be accomplished, when there are several bars, by bending the bars in two or more lines parallel to each other, as is shown in the Cummings' reinforcement, and as is common practice with the ordinary reinforcement. It may be expected, then, that good results may be obtained by bending up part of the bars if proper care is taken to get a good distribution of the diagonals.

33. *Web Resistance of Cummings' Reinforcement.*—As noted on page 54, all the beams reinforced with the Cummings' system of reinforcement failed by tension in the steel. The nominal vertical shearing stress developed was high, averaging 238 lb. per sq. in.; how much greater stress could have been developed is not known. The cracks in the outer thirds of the span length were

well distributed and opened up very little. The principle on which this form of reinforcement is based seems rational,—the main reinforcement is carried horizontally to the end of the beam, the smaller bars are bent up and this bending is made in two planes (i. e., begins at two points), the bars are brought together at their ends and thus are anchored, and the reinforcement is connected by a cross bar at the points where bending up begins, which aids in distributing the web stresses among the bars. It does not seem that welding the bars at the loop is essential, bending them around with sufficient anchorage may give the same effect, and other devices of similar make-up may be expected to give good results. It may be added that a unit frame has many advantages in reinforced concrete construction, one important feature being to insure that the reinforcement is properly placed and is retained in the proper place during fabrication.

C. BEAMS WITH STIRRUPS.

34. *Beams with Stirrups.*—Tests of 43 beams having stirrups for web reinforcement are included in the 1907 series, and 12 beams in the 1908 series. A classification of the beams according to the longitudinal and the web reinforcement used may be made as follows: (a) longitudinal reinforcement,—(1) smooth round bars, (2) deformed bars; (b) stirrups,—(1) smooth round rods, (2) smooth round rods bent at top, (3) smooth round rods having a reduced section, (4) smooth round rods wrapped with paper for a part of their length, (5) corrugated bars of high elastic limit, (6) corrugated bars of mild steel. In addition to these, test beams were made of 1-5-10 concrete to determine the effect of poor concrete. Other beams were tested to study the effect of unsymmetrical loading. The tests were planned with a view of determining the amount of stress (tension and bond) developed in the stirrups. However, for various reasons, the results are of less value than was expected. The beams were not all made according to the plans. In the 1907 tests, the stirrups in a few of the beams were poorly placed and even left exposed at the face of the beam, and a variation in the temperature conditions of the laboratory also affected the results. It is evident from the results that the stresses developed in the stirrups are less than they were calculated to be, and hence the layout was not well planned to settle the points at issue. The tests, however, give considerable information on the

effectiveness of stirrups in providing web resistance.

Tables 13, p. 62 and 64, and 14, p. 63 and 65, give data of the beams and of the tests. The values of tensile stress in stirrups were obtained by equation (21) as described on page 18. The bond stresses developed were calculated as described on page 20, using as the bond surface the surface area of the stirrup for six-tenths of the distance from the top of the beam to the center of the longitudinal reinforcement.

35. *Phenomena of Tests.*—It seems evident from the tests that the stirrups did not take much stress until after the formation of diagonal cracks. Diagonal cracks appeared on the surface of the beam at loads giving the same vertical shearing stress v as those at which cracks appear in beams of the same span, reinforcement, and quality of concrete not having web reinforcement, or at loads somewhat greater. A good illustration of the way in which deformations begin to show in stirrups is given in Bulletin No. 28, "A Test of Three Large Reinforced Concrete Beams." The diagram of vertical readings (Fig. 10, p. 22) shows first a vertical shortening followed by elongation. The point of beginning of marked elongation is at or near the load at which diagonal cracks appeared in the other beam test. It seems evident that there is very little elongation in stirrups until the first diagonal crack forms, and hence that up to this point the concrete takes practically all the diagonal tension. If, after diagonal cracks have formed, the load is released and then reapplied, the stirrups will take stress from the beginning. This is discussed in Bulletin No. 28 already referred to.

After the cracks become visible, the stirrups take tensile and bond stresses, and the diagonal cracks extend and enlarge. Finally the diagonal cracks enlarge considerably. The inability to carry a greater load was essentially a diagonal tension failure in all cases where failure occurred outside the load points. A feature of the tests of beams with stirrups is slow failure, the load holding well up to the maximum under increased deflection and giving warning of its condition.

Not enough information was obtained to determine the actual final occasion of failure in these tests. In a number of cases the stirrups slipped, in others it seemed that the steel in the stirrups was stretched beyond its elastic limit, and in some cases the

TABLE 13.
DATA OF BEAMS WITH STIRRUPS.
Series of 1907.

Beam No.	Kind of Concrete	Cement		Longitudinal Reinforcement		Stirrups	
		Kind	Per cent	Description	Per cent	Description	Spacing
231.1	1-2-4	AA	18.3	4 $\frac{1}{2}$ -in. cor. bars	1.25	$\frac{1}{4}$ -in. cor. h. s.	6
231.5	do.	U	15.8	do.	1.25	do.	6
231.6	do.	U	15.5	do.	1.25	do.	6
232.1	do.	AA	18.3	do.	1.25	$\frac{1}{2}$ in. cor. m. s.	4
232.2	do.	AA	16.3	do.	1.25	do.	4
232.5	do.	U	16.	do.	1.25	do.	4
232.6	do.	U	16.	do.	1.25	do.	4
233.1	do.	AA	18.7	do.	1.25	$\frac{1}{2}$ -in. cor. m. s.	8
233.2	do.	AA	16.3	do.	1.25	do.	8
233.5	do.	U	16.2	do.	1.25	do.	8
233.6	do.	U	13.9	do.	1.25	$\frac{1}{2}$ -in. round	8
233.7	do.	U	15.3	do.	1.25	$\frac{5}{8}$ -in. round	8
235.1	1-5-10	AA	6.4	do.	1.25	$\frac{1}{4}$ -in. cor. m. s.	4
235.2	do.	AA	7.2	do.	1.25	do.	4
235.5	do.	U	6.3	do.	1.25	do.	4
235.6	do.	U	6.7	do.	1.25	do.	4
241.5	1-2-4	U	16.	do.	1.25	$\frac{1}{2}$ -in. round bent	8
241.6	do.	U	15.5	do.	1.25	do. [inward	8
242.1	do.	AA	12.6	4 $\frac{1}{2}$ -in. round	0.98	$\frac{1}{2}$ -in. round	8
242.2	do.	AA	14.9	do.	0.98	do.	8
243.1	do.	AA	15.6	4 $\frac{1}{2}$ -in. cor. bar h. s.	1.25	do.	8
243.2	do.	AA	12.6	4 $\frac{1}{2}$ -in. round	0.98	do.	8
244.1	do.	AA	14.9	do.	0.98	do.	6
244.2	do.	AA	15.6	4 $\frac{1}{2}$ -in. cor. bar	1.25	do.	6
271.5	do.	U	15.9	2 $1\frac{1}{4}$ -in. angles	1.11	Expanded metal	
271.6	do.	U	15.5	do.	1.11	do.	
221.1	do.	AA	15.8	3 $\frac{3}{4}$ -in. round	1.65	$\frac{1}{2}$ -in. round	5
221.2	do.	AA	15.7	do.	1.65	do.	5
221.5	do.	U	15.5	do.	1.65	do.	5
221.6	do.	U	15.2	do.	1.65	do.	5
222.5	do.	U	15.4	5 $\frac{1}{2}$ -in. round	1.23	$\frac{5}{8}$ -in. round re-	5
222.6	do.	U	15.7	4 $\frac{1}{2}$ -in. round	0.98	do. [duced to $\frac{5}{16}$	5
223.1	do.	AA	16.0	do.	0.98	$\frac{1}{4}$ -in. round m. s.	3
223.5	do.	U	15.6	5 $\frac{1}{2}$ -in. round	1.23	do.	3
223.6	do.	U	15.6	do.	1.23	do.	3
224.5	do.	U	15.7	do.	1.23	$\frac{1}{4}$ -in. round re-	3
224.6	do.	U	15.7	4 $\frac{1}{2}$ -in. round	0.98	do. [duced to $\frac{7}{32}$	3
225.5	do.	U	16.6	3 $\frac{3}{4}$ -in. round	1.65	$\frac{5}{8}$ -in. round	7
225.7	do.	U	15.4	4 $\frac{1}{2}$ -in. round	0.98	do.	7
226.5	do.	U	15.6	do.	0.98	$\frac{1}{2}$ -in. round; lower 4	5
226.6	do.	U	15.6	do.	0.98	in. wrapped in paper	5
227.5	do.	U	15.6	3 $\frac{3}{4}$ -in. round	1.65	$\frac{1}{2}$ -in. round	5
227.6	do.	U	15.6	do.	1.65	do.	5
228.5	do.	AA	15.5	do.	1.65	$\frac{1}{4}$ -in. round	3
228.6	do.	U	15.5	do.	1.65	do.	3

(Continued on page 64.)

TABLE 14.
TESTS OF BEAMS WITH STIRRUPS.
Series of 1907.

All beams failed by diagonal tension unless otherwise noted.

Beam No.	Age days	Load at First Diagonal Crack pounds	Maximum Applied Load pounds	Stress in Longitudinal Reinforcement lb. per sq. in.	Vertical Shearing Stress lb. per sq. in.	Bond in Longitudinal Rein. lb. per sq. in.	Tensile Stress in Stirrups lb. per sq. in.	Bond in Stirrups lb. per sq. in.
231.1	60	25 260	33 000	47 800	251	251	96 400	1004
231.5	66	18 000	35 800	51 800	272	272	104 400	1088
231.6	71	20 000	31 800	46 000	242	242	93 000	968
232.1*	60	28 000	38 000	55 000	281	281	72 000	750
232.2*	60	24 000	40 300	58 200	306	306	78 400	815
232.5	62	17 000	22 000	32 000	170	170	43 500	453
232.6	61	19 000	25 000	36 300	192	192	49 100	512
233.1*	61	27 000	43 300	62 500	328	328	42 000	874
233.2	61	20 000	34 700	50 200	264	264	33 800	705
233.5	63	14 000	34 000	49 300	259	259	33 100	690
233.6	61	14 000	29 300	42 500	224	224	36 500	714
233.7	63	23 000	27 200	39 500	208	208	21 600	563
235.1	63	14 500	14 500	21 900	116	116	29 800	310
235.2	61	17 000	17 800	26 700	141	141	36 100	376
235.5†	61	6 100	6 100	9 600	52	52	13 300	139
235.6	61	8 000	11 900	18 100	100	100	25 600	267
241.5	59	18 000	32 300	46 700	246	246	40 200	834
241.6	52	18 000	29 400	42 600	224	224	37 500	760
242.1*	60	21 300	22 500	41 300	171	218	27 900	580
242.2*	60	20 000	24 500	44 800	193	246	31 500	580
243.1	60	26 000	44 300	64 000	336	336	34 900	1140
243.2*	60	24 000	25 000	47 600	197	251	32 400	668
244.1*	60	22 000	24 100	43 300	185	185	22 600	470
244.2	60	24 000	28 200	40 800	216	216	26 500	549
271.5	59	12 000	13 800	22 800	110			
271.6	61	15 000	16 300	26 700	127			
221.1	60	17 000	25 000	28 000	195	221	19 900	413
221.2	58	22 000	26 000	29 100	203	230	20 700	430
221.5	60	20 000	23 000	25 800	181	205	18 500	383
221.6	59	21 000	21 000	23 600	165	187	16 800	350
222.5	61	20 000	23 700	35 000	182	185	47 600	310
222.6	20	17 000	17 200	32 500	132	168	34 400	224
223.1	61	16 000	19 000	35 000	146	186	35 800	372
223.5	61	19 000	22 600	33 400	174	177	42 700	443
223.6	63	4 000	21 000	31 100	162	165	39 700	412
224.5	60	14 000	21 800	32 300	168	171	52 200	427
224.6*	60	18 000	21 200	38 900	164	209	51 000	417
225.5	59	22 000	22 000	34 700	173	196	15 800	411
225.7	59	20 000	21 700	39 800	165	210	15 000	392
226.5	60	19 000	20 100	36 900	154	196	15 700	326
226.6	61	16 000	21 000	38 600	160	204	16 300	339
227.5	63	18 000	20 000	22 500	158	179	16 100	335
227.6	60	15 000	16 400	18 500	130	147	13 300	275
228.5	59	16 000	17 500	19 700	138	156	33 800	351
228.6	60	19 000	21 400	24 000	168	190	41 400	428

* Tension failure. † Appearance indicated poor concrete.

[Continued on page 65.]

TABLE 13.—*Continued.*

DATA OF BEAMS WITH STIRRUPS.

Series of 1908.

Beam No.	Kind of Concrete	Cement		Longitudinal Reinforcement		Stirrups	
		Kind	Per cent	Description	Per cent	Description	Spacing inches
221.3	1-2-4	AA	15.7	3 $\frac{3}{4}$ -in. round	1.65	$\frac{1}{2}$ -in. round	5
221.4	do.	AA	16.2	do.	1.65	do.	5
223.2	do.	AA	15.7	5 $\frac{1}{2}$ -in. round	1.23	$\frac{3}{4}$ -in. round	3
223.3	do.	AA	16.2	do.	1.23	do.	3
229.1	do.	AA	15.3	do.	1.23	$\frac{1}{2}$ -in. round (one end only)	5
229.2	do.	AA	15.2	do.	1.23	do.	5
229.5	do.	AA	14.3	do.	1.23	$\frac{1}{2}$ -in. cor. m. s. (one end only)	5
229.8	1-1 $\frac{1}{2}$ -3	AA	24.5	do. 2 bent [up	1.23	$\frac{1}{2}$ -in. cor. h. s. (one end only bars bent up at other end)	5
233.3	do.	AA	15.9	4 $\frac{1}{2}$ -in. round	0.98	$\frac{1}{2}$ -in. round	8
233.4	do.	AA	15.7	4 $\frac{1}{2}$ -in. cor.	1.25	$\frac{1}{2}$ -in. cor. m. s.	8
240.1	do.	AA	15.3	3 $\frac{3}{4}$ -in. round nuts and washers	1.65	$\frac{1}{2}$ -in. round	5
243.3	do.	AA	15.3	4 $\frac{1}{2}$ -in. cor.	1.25	do.	8

stirrups broke; possibly in some the longitudinal reinforcement slipped. In a number of tests the general racking or breaking up of the concrete was the evident cause of failure. The following notes of the tests are given.

Beam No. 231.1 (See Fig. 16). At 25 000 lb. diagonal crack marked (1) was noted 10 in. to left of left load. At 26 000 lb. cracks appeared in middle third and diagonal crack (2) was noted, and at 28 000 lb. diagonal cracks (3) and (4). At 29 000 and 30 000 lb. cracks marked (5) and (6) noted at left end of beam. At 32 000 lb. metallic noises were heard and diagonal cracks continued to open. Maximum load, 33 000 lb. Load dropped to 14 900 lb. as ultimate failure occurred at (3) and (4); stirrup marked (a) broke, and stirrup marked (b) slipped.

Beam No. 232.1. At 26 000 lb. vertical crack marked (1) appeared. At 28 000 lb. diagonal crack (2) was noted at left end and diagonal cracks (3) and (4) at right end. At 30 000 lb. diagonal crack (6) appeared. At 32 000 lb.

TABLE 14.—*Continued.*

TESTS OF BEAMS WITH STIRRUPS.

Series of 1908.

All beams failed by diagonal tension unless otherwise noted.

Beam No.	Age days	Load at First Diagonal Crack pounds	Maximum Applied Load pounds	Stress in Longitudinal Reinforcement lb. per sq. in.	Vertical Shearing Stress lb. per sq. in.	Bond in Longitudinal Reinforcement lb. per sq. in.	Tensile Stress in Stirrups lb. per sq. in.	Bond in Stirrups lb. per sq. in.
221.3	57	24 000	31 100	34 600	242	274	24 700	512
221.4	70	20 000	21 170	23 800	166	188	16 900	351
223.2	57	22 000	24 600	36 300	189	192	46 400	481
223.3	70	20 000	26 150	38 500	201	205	48 300	511
229.1†	63	20 000	22 000	43 500	255	259	26 000	540
229.2*†	92	20 000	25 400	50 000	292	296	29 800	618
229.5	72	13 000	13 400	20 100	159	164	12 750	337
229.8	60	14 000	26 100	37 700	200	204	16 000	423
233.3‡	72	18 000	20 850	38 300	159	203	26 000	539
233.4	66	18 000	26 300	38 200	201	201	25 800	536
240.1*	64	26 000	34 800	38 800	270	306	27 500	572
243.3	64	20 000	34 200	49 500	260	260	42 400	880

* Tension failure.

† Load applied at a single point 2 ft. from one support.

‡ Failure at the end not having stirrups.

crack marked (7) was noted at left end and vertical cracks marked (8) and (9) in middle third, and at 34 000 lb. vertical crack (10). At 36 000 lb. tension cracks in middle third were opening, diagonal cracks not opening much. Maximum load, 38 000 lb. Load dropped to 32 600 lb. failure occurring by tension in steel.

Beam No. 232.2. At 24 000 lb. numerous vertical cracks noted in middle third and diagonal cracks marked (1) at left end, but these did not open very wide. At 32 000 lb. diagonal cracks marked (2) were noted at right end. Maximum load, 40 300 lb. Failure occurred by tension in steel.

Beam No. 232.6. At 19 000 lb. a short diagonal crack marked (1) noted at right end at mid-depth of beam. At 20 000 lb. the above crack extended downward to bottom of beam and upward along fifth stirrup from end, a branch extending to load point. At 21 000 lb. crack marked (2) noted. Failure occurred by diagonal tension. Maximum load, 25 000 lb.

Beam No. 233.1. At 27 000 lb. a diagonal crack marked (1) was noted 21 in. to right of center and extended vertically 6 in. At 36 000 lb. a diagonal crack marked (2), 6 in. high, was noted 18 in. to left of center

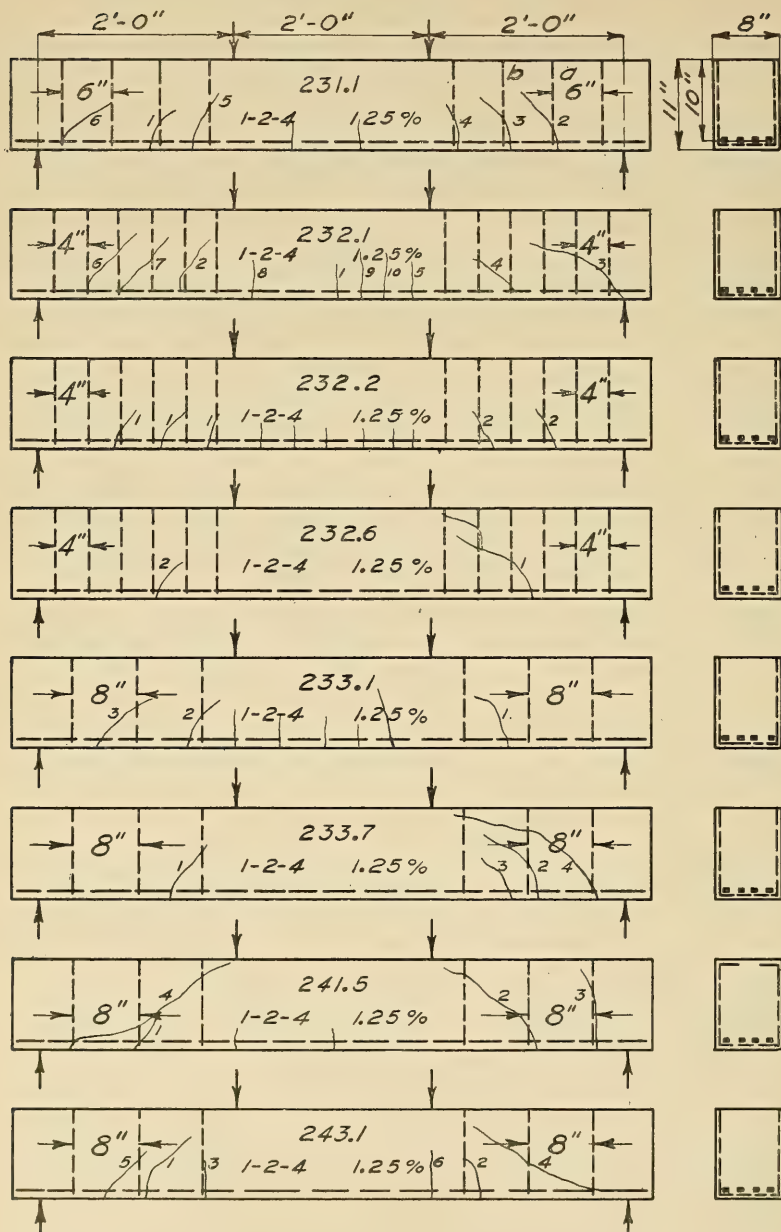


FIG. 16. DISPOSITION OF REINFORCEMENT AND POSITION OF CRACKS.

of beam. At this load a vertical crack 5 in. long appeared under left load, tension cracks 4 in. long were noted $6\frac{1}{2}$ and 1 in. to left of center and a tension crack 3 in. high 3 in. to right of center. At 41 000 lb. a diagonal crack marked (3) extended vertically 5 in. At 43 200 lb. all cracks were opening. At maximum load, 43 300 lb. a tension crack 7 in. long noted 7 in. to right of center. Failure occurred by tension in steel.

Beam No. 233.2. At 20 000 lb. a diagonal crack appeared 9 in. to left of left load and extended vertically 4 in. At 28 000 lb. several vertical cracks were noted in middle third, and two diagonal cracks at right end. Diagonal crack noted at left end at load of 32 000 lb. Maximum load 34 700 lb. Failure occurred at right end by diagonal tension probably followed by slipping of bars. Middle stirrup at right end finally split out at near the maximum load on account of being too near the surface—about 1 in. center to surface.

Beam No. 233.7. At 23 000 lb. a diagonal crack marked (1) noted which extended vertically 5 in. At 24 000 lb. a diagonal crack marked (2), 7 in. high, noted. At 26 000 lb. diagonal cracks marked (3) and (4) appeared and extended vertically 4 in. The other cracks were opening up at this load. At 27 000 lb. the cracks were lengthening. At 27 200 lb., the maximum load, diagonal crack marked (4) opened and extended to load point. Load dropped to 22 000 lb., failure occurring by diagonal tension with slipping of stirrups.

Beam No. 235.1. At the maximum load of 14 500 lb. a diagonal crack 6 in. high appeared 13 in. to left of left load. Load dropped gradually, and at 13 000 lb. the crack was opening up rapidly toward load point.

Beam No. 235.5. Mixture was lean and concrete seemed poorly rammed at bottom. Failure occurred by diagonal tension. On one side the diagonal crack was observed at third stirrup from load. This stirrup was afterward found to be broken. Maximum load 6100 lb.

Beam No. 241.5. Stirrups bent in at top. At 18 000 lb. diagonal cracks marked (1) and (2) noted, (2) extending toward load point at 27 000 lb. At 22 000 lb. a crack 10 in. long marked (3) appeared. At 26 000 lb. a crack marked (4) was noted at left stirrup 2 in. above bottom of beam, extending to second stirrup from left end, and from there to load point. Maximum load 32 300 lb., failure occurring by diagonal tension.

Beam No. 243.1. At 26 000 lb. diagonal cracks marked (1) and (2) appeared. At 30 000 lb. a diagonal crack 6 in. high marked (4) noted at right end, 13 in. from load. At 32 000 lb. the cracks at left end were opening wider. At 34 000 lb. a diagonal crack marked (5) was noted 16 in. to left of left load and extended upward 6 in. Maximum load 44 300 lb., failure occurring by diagonal tension.

Beam No. 244.1. (See Fig. 17). At 18 000 lb. vertical cracks 4 in. long marked (1) noted $2\frac{1}{2}$ in. to right of right load and $2\frac{1}{2}$ in. to left of left load at stirrups. At 22 000 lb. a diagonal crack marked (2) appeared $11\frac{1}{2}$ in. to left of left load and extended vertically $4\frac{1}{2}$ in. At 23 700 lb. the crack at north end was opening wide. At 24 000 lb. a vertical crack marked (3), 6 in. high, was noted 3 in. to the left of center. Maximum load 24 100 lb. Tension crack marked (1) to left of load opened wide after the maximum was reached.

Beam No. 271.6. At 12 000 lb. vertical crack 5 in. long was noted 5 in. to right of right load. A short diagonal crack was also noted in lower third of beam on opposite side 7 in. to right of right load. At 13 800 lb. failure crack appeared suddenly, starting at right end 7 in. inside the support and extending to within 4 in. of load. The expanded metal web was not sufficient to give much resistance.

Beam No. 221.2. Three stirrups at right end were showing at surface. Concrete was poor. At 22 000 lb. a diagonal crack marked (1), 4 in. high, was noted 12 in. to right of left support. At 24 000 lb. diagonal crack

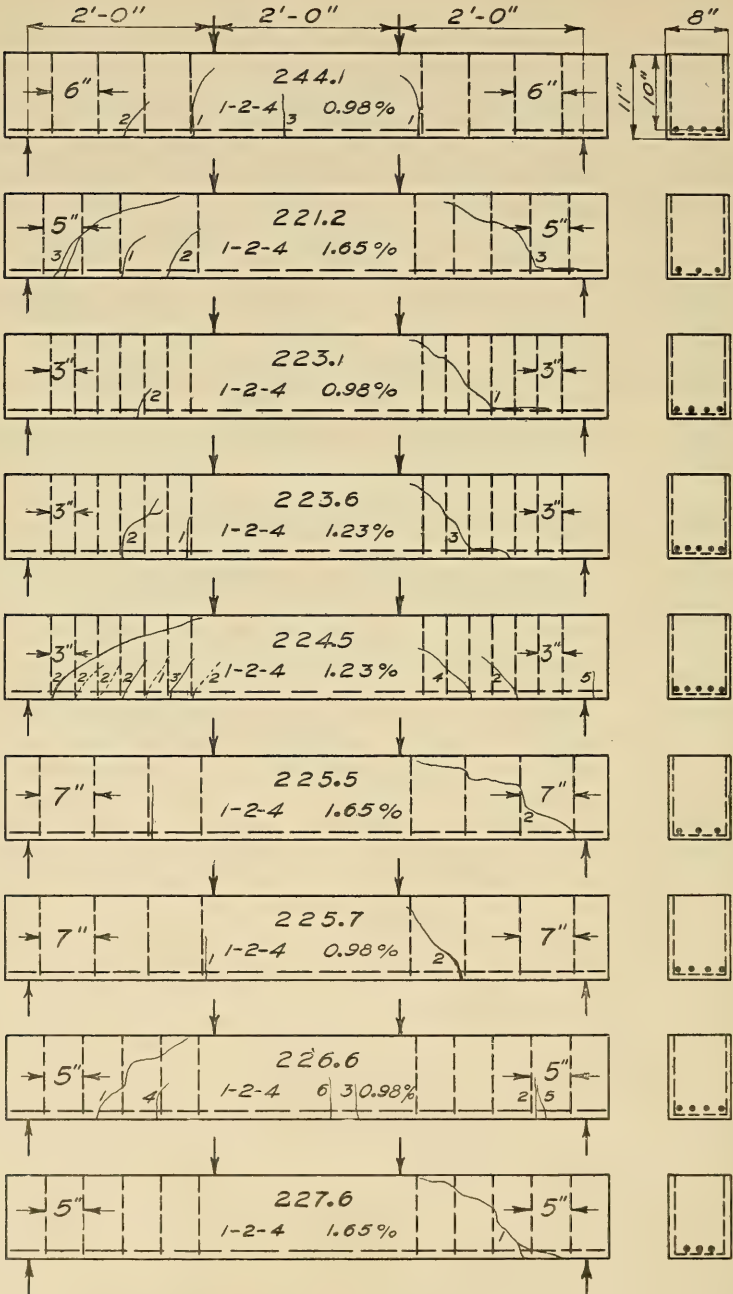


FIG. 17. DISPOSITION OF REINFORCEMENT AND POSITION OF CRACKS.

5 in. high marked (2), 6 in. to left of left load, was noted. At 26 000 lb. diagonal crack marked (3) appeared. Maximum load 26 000 lb., diagonal tension failure.

Beam No. 222.5. At 16 000 lb. vertical crack was noted 6 in. to left of left load. At 20 000 lb. one 5 in. to left of center and at 22 000 lb. 3 in. to right of center. At 20 000 lb. diagonal cracks were noted 10 in. to left of left load and $3\frac{1}{2}$ in. to right of right load. Failure occurred by diagonal tension. Maximum load 23 700 lb. All the concrete broke away from bottom of second stirrup to left of left load. Reduced section could be plainly seen. Stirrup did not slip.

Beam No. 223.1. At 16 000 lb. diagonal crack 3 in. high marked (1) was noted. At 18 000 lb. this crack was 5 in. high and diagonal crack 4 in. high marked (2) was noted. Crack marked (1) opened wider at maximum load of 19 000 lb. and extended almost to load point. Failure by diagonal tension at right end.

Beam No. 223.5. At 19 000 lb. diagonal cracks were observed 8 in. to left of left load and 8 in. to right of right load. At 22 000 lb. a diagonal crack was noted 14 in. to right of right load. Maximum load 22 600 lb. Failure occurred by diagonal tension and failure crack was more nearly vertical than usual. Failure crack was limited to space between last three stirrups nearest the load. A loose stirrup was noted 6 in. to left of left load.

Beam No. 223.6. Diagonal cracks were noted 12 in. to left of left load and 9 in. to right of right load. Crack at right end extended almost to load point. Failure by diagonal tension at load of 21 000 lb.

Beam No. 224.5. At 14 000 lb. diagonal crack marked (1) was noted, at 15 000 lb. (2), at 16 000 lb. (3), and at 17 000 lb. crack (4). The cracks which started from the bottom of the stirrups headed for the center point of next stirrup toward load. Failure occurred by diagonal tension along diagonal crack at left end and several stirrups slipped at this end.

Beam No. 225.5. At 18 000 lb. crack marked (1) appeared. Failure occurred by diagonal tension along crack marked (2). Second stirrup from right end bulged out and drew down. Maximum load 22 000 lb.

Beam No. 225.7. At 17 000 lb. vertical crack 4 in. high marked (1) was noted and at 20 000 lb. diagonal crack 4 in. high marked (2). At 26 900 lb., the maximum load, the crack marked (2) extended almost to load point. Load dropped to 16 000 lb. Failure occurred by diagonal tension. Concrete around second stirrup to right of right load was very lean. Lower part of the stirrup on each side was exposed and stirrup slipped a little.

Beam No. 226.5. At 18 000 lb. the vertical crack 5 in. high was noted at stirrup 7 in. to left of left load. At 19 000 lb. a diagonal crack 4 in. high was noted 15 in. to right of right load. At 20 000 lb. the cracks were enlarging and at maximum load of 20 100 lb. a diagonal crack 6 in. high was noted 8 in. to right of right load. Load dropped to 11 300 lb.

Beam No. 226.6. At 16 000 lb. diagonal crack 3 in. high marked (1) was noted, at 18 000 lb. vertical crack along stirrup 5 in. high marked (2) and at 19 000 lb. the cracks marked (3) and (4). At 20 000 lb. diagonal crack marked (5) appeared and other cracks were enlarging. At maximum load 21 000 lb., vertical crack marked (6) was noted; diagonal cracks were lengthening. Load dropped to 14 000 lb. and diagonal crack (1) was opening. Failure by diagonal tension at left end. The stirrups were quite close to sides of beam and were wrapped in paper for the lower 4 inches.

Beam No. 227.5. At 18 000 lb. small vertical cracks were noted 2 in. to right of right load, $4\frac{1}{2}$ in. to left of left load, and at first stirrup 2 in. to left of left load on further side of beam. The latter crack was 4 in. long. Failure crack started at bottom of second stirrup 7 in. to right of right load, ran up stirrup $2\frac{1}{2}$ in., then to first stirrup at a point $3\frac{1}{2}$ in. from top then upward along the stirrup. Maximum load 20 000 lb. Two stirrups were exposed at left end.

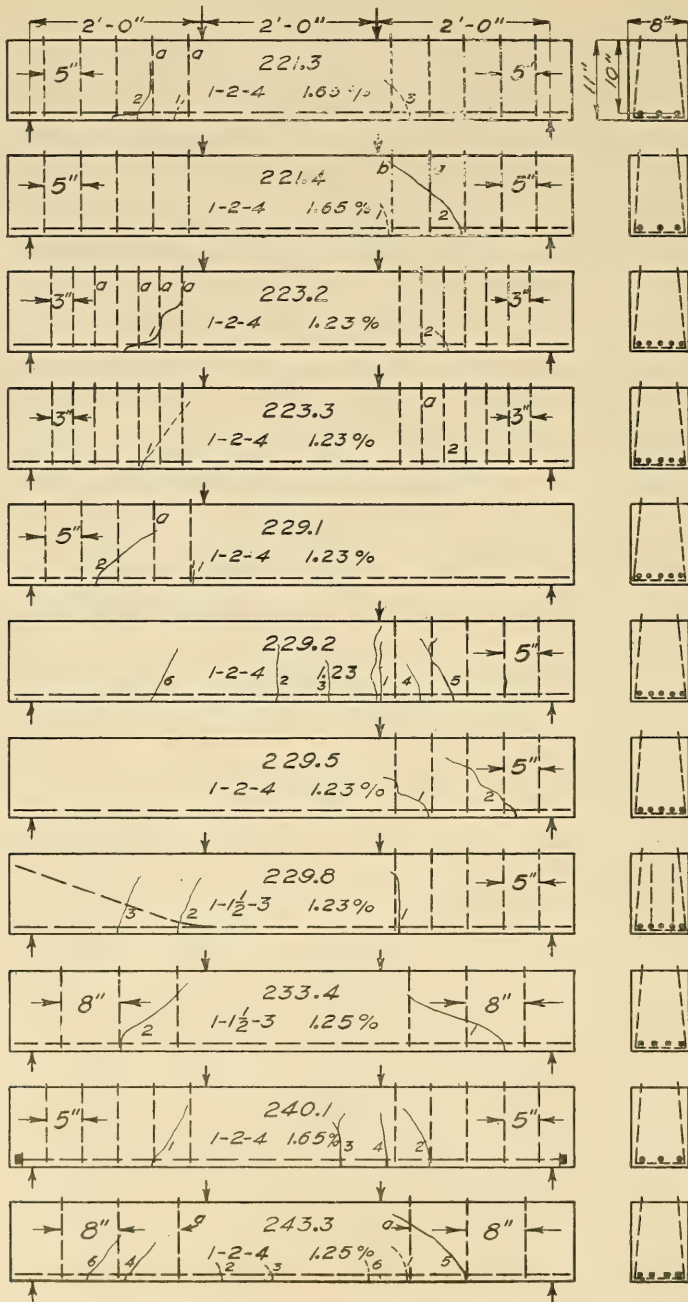


FIG. 18. DISPOSITION OF REINFORCEMENT AND POSITION OF CRACKS.

Beam No. 227.6. Cracks were noted on both sides of beam at 15 000 lb. No cracks noted at left end of beam. Failure was rather sudden and was due to diagonal tension. Third and fourth stirrups from right end were found to have slipped $\frac{1}{2}$ or $\frac{3}{4}$ in. The stirrups were not well embedded, especially at north end where failure occurred. Maximum load 16 400 lb.

Beam No. 228.5. At 16 000 lb. a diagonal crack $5\frac{1}{2}$ in. high was noted 3 in. to left of left load. At maximum load, 17 500 lb., a diagonal crack $5\frac{1}{2}$ in. high was noted 6 in. to right of right load. Failure occurred along the diagonal crack noted at 16 000 lb. The crack ended at bottom of beam at a point near first stirrup to left of load. First two stirrups at left end on further side of beam were showing at bottom. This fact did not affect the results, however.

Beam No. 228.6. At 19 000 lb. diagonal crack was noted 8 in. to right of right load on further side of beam and extended upward 4 in. At 20 000 lb. diagonal crack was noted 12 in. to left of left load and extended toward load point, starting at fourth stirrup from left end. One stirrup showed slightly at right end. Maximum load 21 400 lb., failure occurring by diagonal tension.

Beam No. 221.3. (See Fig. 18). At 24 000 lb. crack marked (1) noted at 27 000 lb. cracks marked (2) and (3), each being about 3 in. high. At 29 000 lb. cracks were lengthening. At 31 000 lb. the two stirrups marked (a) slipped as crack (2) opened. After the maximum load of 31 100 lb. the load fell off to 18 400 lb. and failure crack gradually opened.

Beam No. 221.4. At 20 000 lb. crack marked (1) noted. At 21 000 lb. stirrup marked (a) slipped. At maximum load, 21 170 lb., stirrup marked (b) slipped. Load fell off and crack gradually widened. Diagonal tension failure.

Beam No. 223.2. At 22 000 lb. crack marked (1) noted. At 24 000 lb. this crack was 7 in. high and crack marked (2) was noted. Maximum load 24 600 lb. Gradual failure by diagonal tension. The load was taken off and then applied again. It rose to 13 300 lb. then fell off to 12 200 lb. as failure crack opened and stirrups marked (a) slipped.

Beam No. 223.3. At 20 000 lb. crack marked (1) noted and at 23 000 lb. this crack was $7\frac{1}{2}$ in. high. Maximum load 26 150 lb. Failure was gradual and stirrup marked (a) slipped after beam started to fail.

Beam No. 229.1. Loaded at one point. At 20 000 lb. crack marked (1) noted 3 in. high. At 22 000 lb. diagonal failure crack marked (2) opened and stirrup marked (a) showed signs of slipping. Load dropped to 17 000 lb.

Beam No. 229.2. Also loaded at one point. At 14 800 lb. vertical crack marked (1) noted. At 18 000 lb. cracks marked (2), (3) and (4) noted. At 20 000 lb. crack marked (5) appeared and other cracks lengthened. At 24 200 lb. crack under load was 8 in. long and $\frac{1}{16}$ in. wide. Maximum load 25 400 lb., failure occurring by tension in steel. This shows a high resistance for the portion of the beam without stirrups.

Beam No. 229.5. Also loaded at one point. At 13 000 lb. crack marked (1) noted. At 13 400 lb., the maximum load, crack marked (1) was 4 in. high, and crack marked (2) was noted. Load fell off to 12 000 lb., failure occurring by diagonal tension. No signs of stirrups slipping and stirrups were not exposed.

Beam No. 229.8. Stirrups at one end only; at other end two bars bent up. At 14 000 lb. crack marked (1) was noted. At 20 000 lb. cracks marked (2) and (3) were noted, the former being 4 in. high and the latter 8 in. Cracks (1), (2) and (3) increased in length regularly as load was applied, and at 26 000 lb. cracks (2) and (3) were about 9 in. long and crack (1) 11 in. Crack (1) then widened appreciably. Maximum load 26 100 lb. Failure occurred at the end having stirrups and took place slowly. Whether slip of stirrup along longitudinal bar or slip of the latter occurred is not known. Concrete was rich mixture.

Beam No. 233.4. At 18 000 lb. cracks marked (1) and (2) were noted. Up to maximum load of 26 300 lb. the cracks gradually lengthened and failure occurred by diagonal tension.

Beam No. 240.1. This beam had nuts and washers at ends of longitudinal rods. At 26 000 lb. cracks marked (1) and (2) appeared. At 33 000 lb. cracks marked (3) and (4) were noted and other cracks were lengthening. Failure occurred by tension in steel at 34 800 lb. followed by crushing of concrete. Stirrups did not slip.

Beam No. 243.3. At 20 000 lb. crack marked (1), 3 in. high, appeared. At 21 000 lb. crack marked (2) noted near left load point. At 23 000 lb. crack marked (3), 3 in. high, appeared. At 26 000 lb. diagonal crack marked (4), $4\frac{1}{2}$ in. high noted at left end of beam. Crack marked (1) was 7 in. high at this load. At 30 000 lb. crack marked (5) was noted. At 34 000 lb. cracks marked (6) and (7) were noted, the other cracks were lengthening and at this load stirrups marked (a) had slipped. Maximum load was 34 200 lb., failure occurring by diagonal tension.

36. *Stresses in Stirrups.*—The assumption in the calculations of an effective bond area given by six-tenths of the length of the stirrup is arbitrarily made, but it gives a common basis of comparison. It will be seen in Table 14 that the tensile stresses in the stirrups resulting from this calculation are generally low, though in some cases they run well above the yield point of the steel, and even above its ultimate strength. Thus, the calculated stresses in the high carbon corrugated bar stirrups used in No. 231.1, 231.5 and 231.6 (93 000 to 104 000 lb. per sq. in.) and the stresses in the mild steel corrugated bar stirrups used in No. 232.1 and 232.2 (72 000 and 78 000 lb. per sq. in.) are higher than the elastic limit and ultimate strength of the steel. The bond stresses in many cases are also much above what would be expected to cause slip. These calculated results, and other tests obtained in this laboratory and elsewhere seem to corroborate the view expressed on page 19 that the stress actually developed in stirrups is less than that calculated by equation (21) and that a part of the shear is carried through the concrete of the top of the beam. The actual proportion taken by the stirrups has not been established, but it seems that the stress can not be more than two-thirds or three-fourths of the amount obtained by the formula. On this basis the calculation for tension and bond in stirrups would be made by using say two-thirds of the external vertical shear instead of the full amount.

37. *Comparison of Results.*—As already stated, slip of stirrups and insufficient bond resistance were in many cases the immediate cause of diagonal tension failures, and therefore bond resistance

of stirrups may be considered a critical stress. If we omit special conditions like poor concrete, exposed stirrups, and unsymmetrical loading the following statements of stresses calculated by the methods already described may be made. The 1907 tests with longitudinal reinforcement of plain round rods and with smooth stirrups gave values of the vertical shearing stress v ranging from 154 to 203 lb. per sq. in. and bond values in the longitudinal reinforcement from 196 to 230 lb. per sq. in. The calculated tensile stress in the stirrups ranged from 15 000 to 52 000 lb. per sq. in., and the calculated bond in the stirrups from 326 to 443 lb. per sq. in. With deformed bars as longitudinal reinforcement and with smooth stirrups, the vertical shearing stress v developed was 208, 216 and 336 lb. per sq. in. and the bond developed in the longitudinal bars 208, 216 and 336 lb. per sq. in. The calculated stress in the stirrups ranged from 21 600 to 36 500 lb. per sq. in., and the bond developed in the stirrups 563 lb. per sq. in. Where bent stirrups were used, the value of the vertical shearing stress v was 246 and 224 lb. per sq. in., the bond developed in the longitudinal bars 246 and 224 lb. per sq. in., and the bond in the stirrups 834 and 760 lb. per sq. in. In beams with stirrups made of corrugated bars, the vertical shearing stress developed ranged from 170 to 328 lb. per sq. in., the calculated tensile stress in the stirrups reached 104 000 lb. per sq. in., and the bond 1088 lb. per sq. in.

In the 1908 tests, in beams with plain longitudinal reinforcement and with smooth stirrups the vertical shearing stress developed was 166 to 292 lb. per sq. in. and the bond in the longitudinal bars 188 to 296 lb. per sq. in. Calculations give a tensile stress as high as 48 000 lb. per sq. in., and bond stresses of 351 to 618 lb. per sq. in., in the stirrups. In the beam with corrugated stirrups the vertical shearing stress developed was 200 lb. per sq. in., and in the beam having two longitudinal bars bent up at one end and having corrugated stirrups at the other, 200 lb. per sq. in. vertical shearing stress was developed and 423 lb. per sq. in. bond stress in the stirrups. In No. 243.3, having longitudinal reinforcement of deformed bars and smooth stirrups, the vertical shearing stress was 260 lb. per sq. in. and the bond stress in the stirrups 880 lb. per sq. in. The 1908 beams were made of somewhat better concrete and the results with the smooth stirrups are as high as are to be expected. No. 240.1 (with nuts and washers on the ends of

the longitudinal reinforcement) gave a vertical shearing stress of 270 lb. per sq. in., which is higher than the results of similar beams without anchorage. Stirrups with bent-in ends have many advantages. Much may be said, also, in favor of deformed bars for stirrups, though of course it is plain that mild steel is best for this purpose. Attention is called to the fact that a value of 296 lb. per sq. in. for the calculated bond resistance with plain round rods as longitudinal reinforcement was obtained in one beam.

38. *Effect of Poor Concrete.*—Beams No. 235.1, 235.2, 235.5 and 235.6, made of 1-5-10 concrete, were tested to study the effectiveness of stirrups with poor concrete. The results vary greatly. No. 235.2 sustained a fair load, but No. 235.5 which gave evidence of very poor concrete, proved to be very weak. It is evident that after the concrete begins to crack badly through diagonal tension, the stirrups are ineffective in giving web strength. There was no evidence of slip of stirrups in these beams.

It was also evident in beams of 1-2-4 mix that quality of concrete affects the effectiveness of stirrups. In sets containing the same arrangement of reinforcing, the higher values obtained are in beams of stronger concrete, as shown by the appearance of the concrete or by auxiliary tests.

However, even with the very lean mixtures the failure of the beam was slow and quite in contrast with the failures of beams of the same mixture not provided with web reinforcement.

39. *Effect of Unsymmetrical Loading.*—Beams No. 229.1, 229.2, 229.5 and 229.8 were made with stirrups in one end of the beam but with none in the other, and the first three were tested with a single concentrated load applied at 2 feet from the support next to the end having stirrups. This arrangement was planned to learn the effect of unsymmetrical loading on beams without web reinforcement, as when a concentrated load is placed at one end of the beam and there are no stirrups in the middle portion, and also to get information on the action of beams with web reinforcement and unsymmetrical loading. No failures occurred at the middle or at the end not having stirrups, although the stresses developed (one-half of the vertical shearing stress given in the table) in the middle and unstirruped end of the beam ranged as high as those found at failure in the series without web reinforce-

ment. It seems evident that concentrated, unsymmetrical loading gives results comparable with those found by symmetrical loading.

In Beam No. 229.8 the end which was not provided with stirrups had instead two bars bent up as shown in Fig. 18. Failure occurred near one stirrup, the crack being nearly vertical. As this was rich concrete, and the concrete did not break up, and as the stresses in the stirrups were small, it would seem as if slip of longitudinals may have contributed to failure. It is classed, however, as a diagonal tension failure. In this beam the web resistance given by stirrups and by bent rods appears to be nearly the same.

40. *Proportioning of Stirrups.*—It is evident that some form of web reinforcement may well be used whenever the value of the vertical shearing stress under working conditions is more than one-third or one-fourth of the values given on page 46 for the quality of concrete assured by the conditions of construction, and also in beams developing even lower stresses if there is an uncertainty in the web strength or in the quality of concrete or if the method of applying the load or the nature of the structure makes such a safeguard desirable. The effectiveness of stirrups in providing additional web resistance is brought out in the tests. The insurance of slow failure given by stirrups is itself an advantage. The spacing of the stirrups should not be more than three-quarters or eight-tenths of the depth of the beam.

From the tests and discussion, it appears that as a tentative method it is allowable to consider that for good concrete one-third of the vertical shear is carried through the concrete in the upper part of the beam and two-thirds go to produce stresses in the stirrups. Using then, two-thirds of the total vertical shear as the V of equation (21), p. 18, the ordinary working stresses for tension and bond may be used in the calculations for stirrups. With these assumptions, a length equal to six-tenths of the depth of the beam may be used in the calculations for bond.

Even for well proportioned web reinforcement the use of high values of v is to be discouraged, and strengths should not be accepted unless tests of the same class of beams made under the same conditions of construction give correspondingly high strengths. It may be expected, too, from analogy to the results in

beams without web reinforcement, that when stirrups are used, stiff beams may be counted on better to resist web stresses than those with lighter reinforcement.

Tests with a combination of stirrups and of bending up of part of the reinforcing bars were not made but it is evident that such an arrangement, say to bend up one-half of the bars, gives added web resistance. The proportion of the diagonal tension taken by the bent-up bars and by the stirrups can not be calculated. In such cases the bond developed in the longitudinal reinforcement should be taken into consideration and it is quite possible that the bond resistance obtained with deformed or anchored bars may be advantageous. There is need of experimental work along these lines.

41. *Web Reinforcement of Expanded Metal.*—It was not expected that Beams No. 271.5 and 271.6 would carry higher loads than beams without web reinforcement, since the amount of material in the web was quite inadequate for web stresses and since the metal was not well placed. The beams are included in the list, however, as an example of the inefficient web reinforcement which has been proposed or used without either experimental or analytical verification of its value.

D. GENERAL COMMENTS ON WEB RESISTANCE

42. *General Comments.*—No effort will be made to review the discussion or summarize results, but a few general statements may be made.

Failures by diagonal tension in beams without web reinforcement are especially objectionable, since, as in the case of tension tests of concrete, the failure may occur without warning. Besides, although the effect of repeating the application of the load after a diagonal crack has formed has not been established, final failure undoubtedly will occur after many applications at a load much lower than that which would cause failure on first application.

In beams without web reinforcement, web resistance depends upon the quality and strength of the concrete. Cement makes a simple and effective strengthening agent. The effect of quality and strength of concrete is very marked. Even where web reinforcement is used, quality of concrete is an important element.

The stiffer the beam the larger the vertical shearing stress which may be developed. Short, deep beams give higher results than long, slender ones, and beams with a high percentage of reinforcement than beams with a small amount of metal. This difference may be due to the relation of diagonal tension to shearing stress, to the difference in opportunity for formation of vertical tension cracks, or to such a cause as a difference in amount of distortion of section.

As beams ordinarily have less than 1% reinforcement, the web strength of such beams may be expected to be less than that of the beams herein described.

There is a considerable increase of web strength at the earlier ages, but not much additional web strength may be counted on under laboratory conditions after 60 days. Care should be exercised that ample time is given for beams to acquire sufficient web strength before loads are applied.

The loads carried by beams with all bars bent up or inclined in the outer thirds of their length did not differ much from those with the bars all straight, but the failure is slower and warning is given. The formation of a vertical crack and the splitting of the concrete along the inclined part of the bar are characteristic of the failures.

Beams with inclined bars firmly anchored at their ends gave high web resistance and showed indications of arch action. This form of construction seems to give insurance against failure at low loads through defective concrete or insufficient bond.

The arrangement of bending up part of the bars and leaving part straight developed a good web strength and secured slow failures. This method will be particularly satisfactory when the number of bars is large enough to permit the spacing of the inclined bars at frequent intervals through the part of the beam where high web resistance is needed. A proper distribution of the diagonals is important.

The Cummings' reinforcement gave high web strength. The distribution of the diagonals fills the requirements above mentioned, a lateral connection is provided, and the main bars are carried to the end of the beam.

Beams provided with U-shaped stirrups which passed under straight reinforcing bars generally gave high web resistance, and

slow failures were characteristic of the tests. The importance of quality of concrete is also apparent in these tests. Adequate bond resistance is essential. The amount of web resistance which may be developed even with carefully arranged stirrups is limited, the limit depending upon the quality of the concrete.

Stirrups do not come into action, at least not to any great extent, until a diagonal crack has formed. After such a crack has formed, the stirrups will take stress from the beginning on a second application of the load.

The tests and calculations go to show that under the maximum loads applied to the beams the stirrups are not stressed to an amount necessary to take the entire vertical shear. The use of a fractional part of the total vertical shear like two-thirds for application in the formulas seems to be warranted.

Considerable variation in results in beams of the same make-up was found. This is to be expected in web resistance. An even greater variation is probable under conditions of practice.

Low working stresses in web resistance are to be commended and ample provision for web strength should be made.

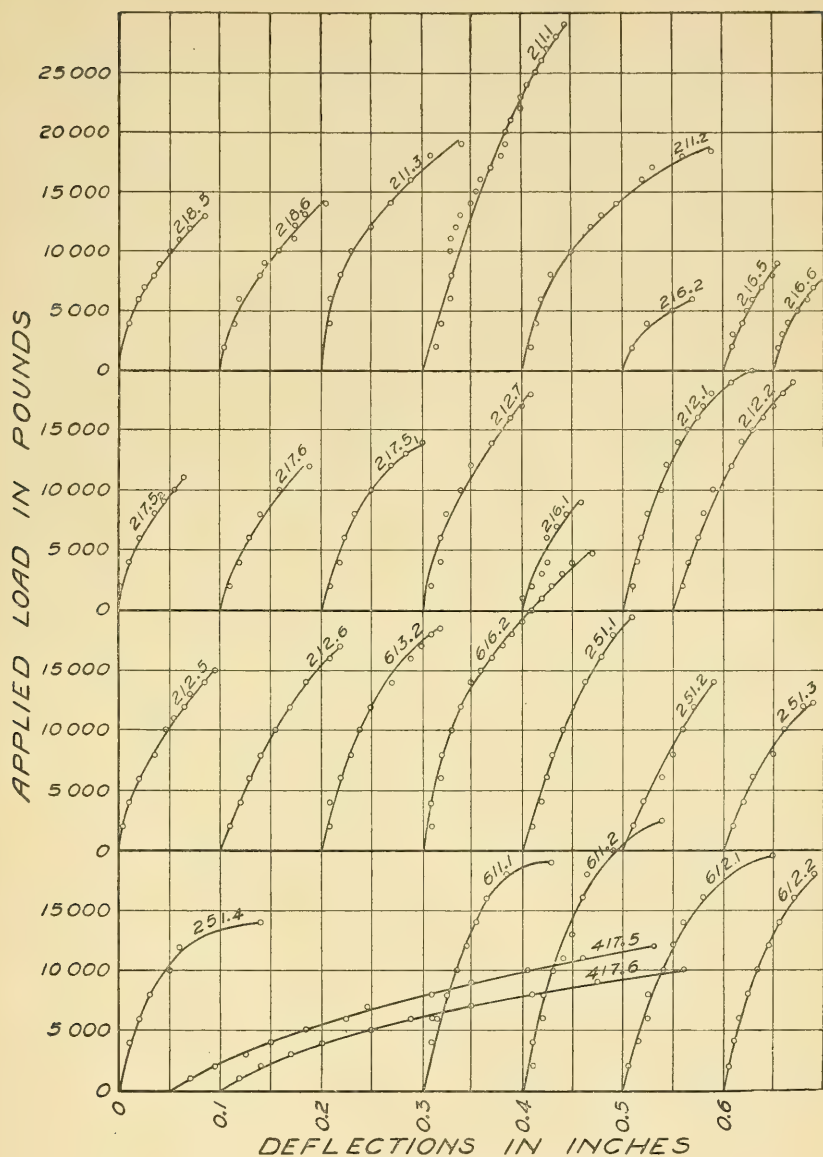


FIG. 19. LOAD-DEFLECTION DIAGRAMS.

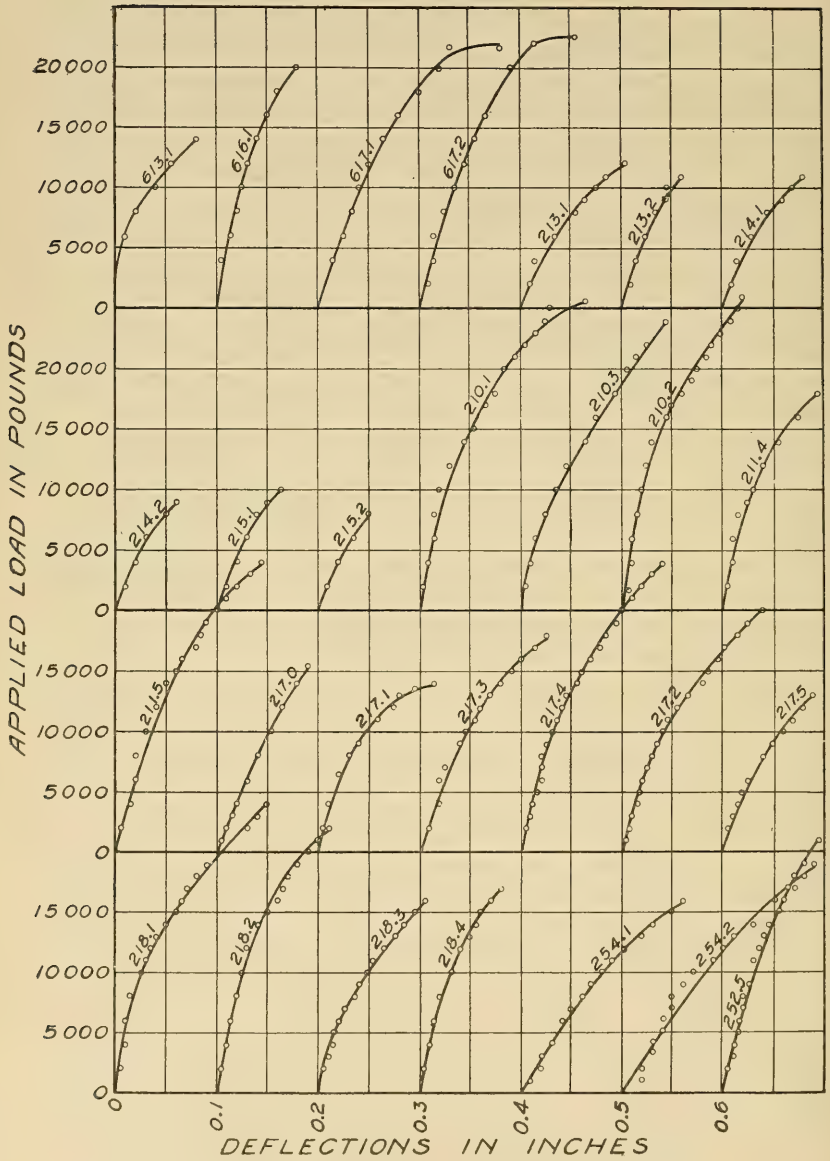


FIG. 20. LOAD-DEFLECTION DIAGRAMS.

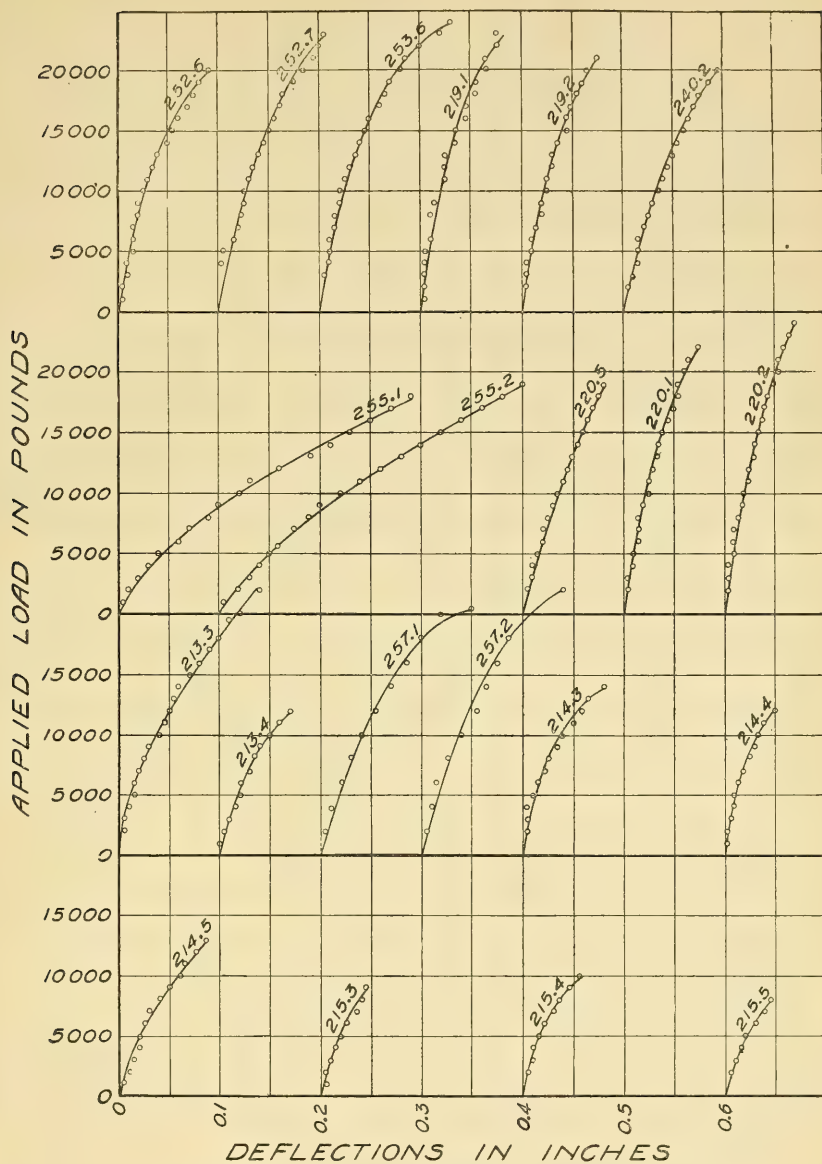


FIG. 21. LOAD-DEFLECTION DIAGRAMS.

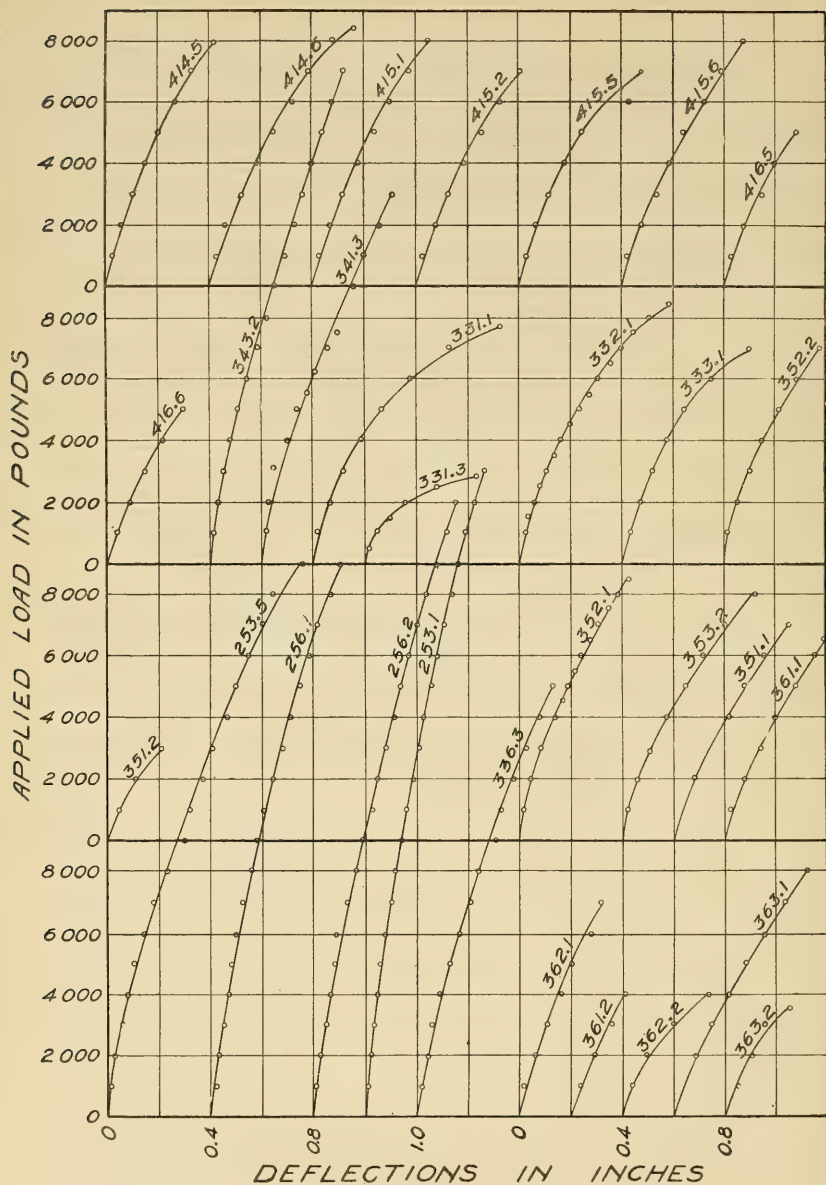


FIG. 22. LOAD-DEFLECTION DIAGRAMS.

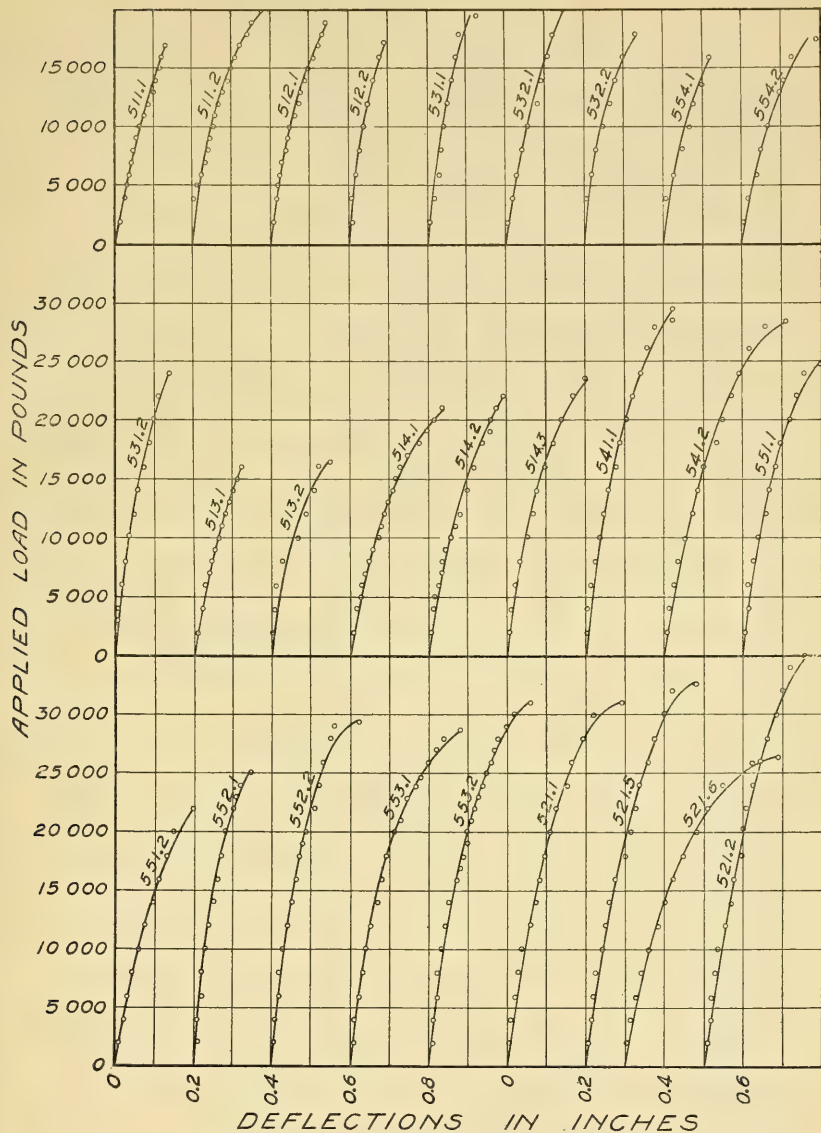
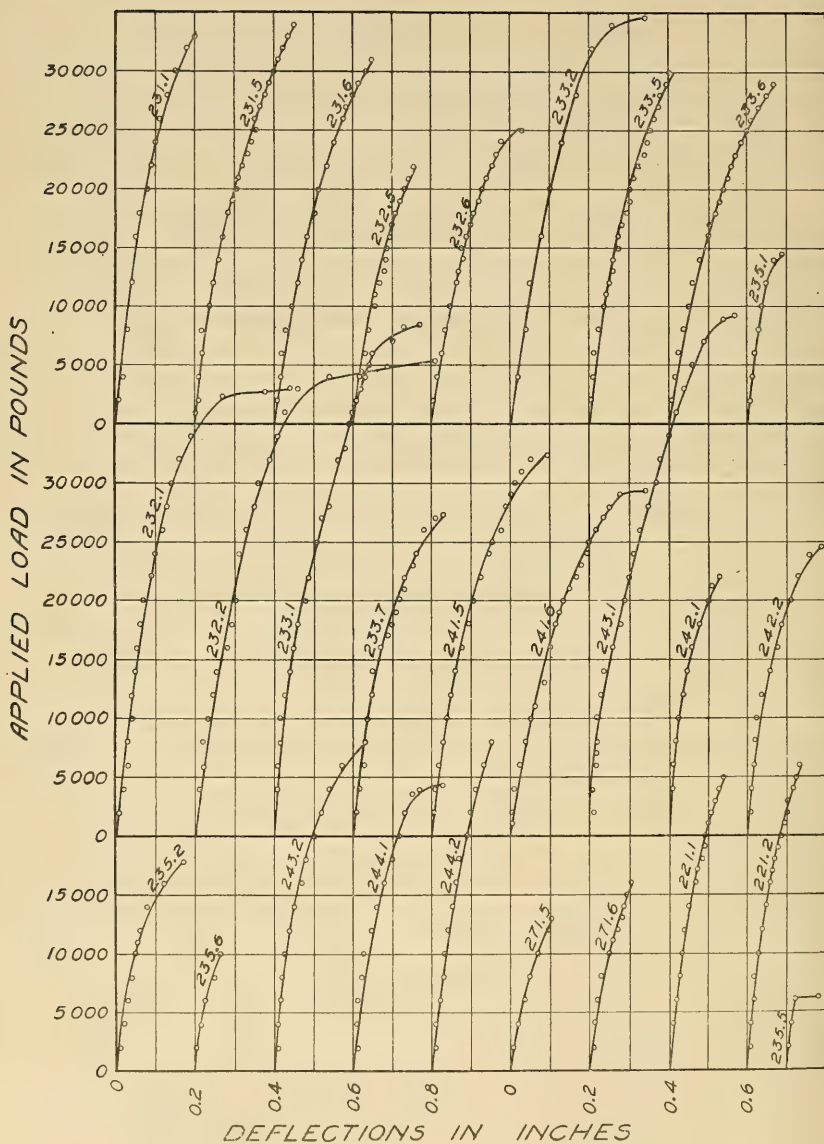


FIG. 23. LOAD-DEFLECTION DIAGRAMS.



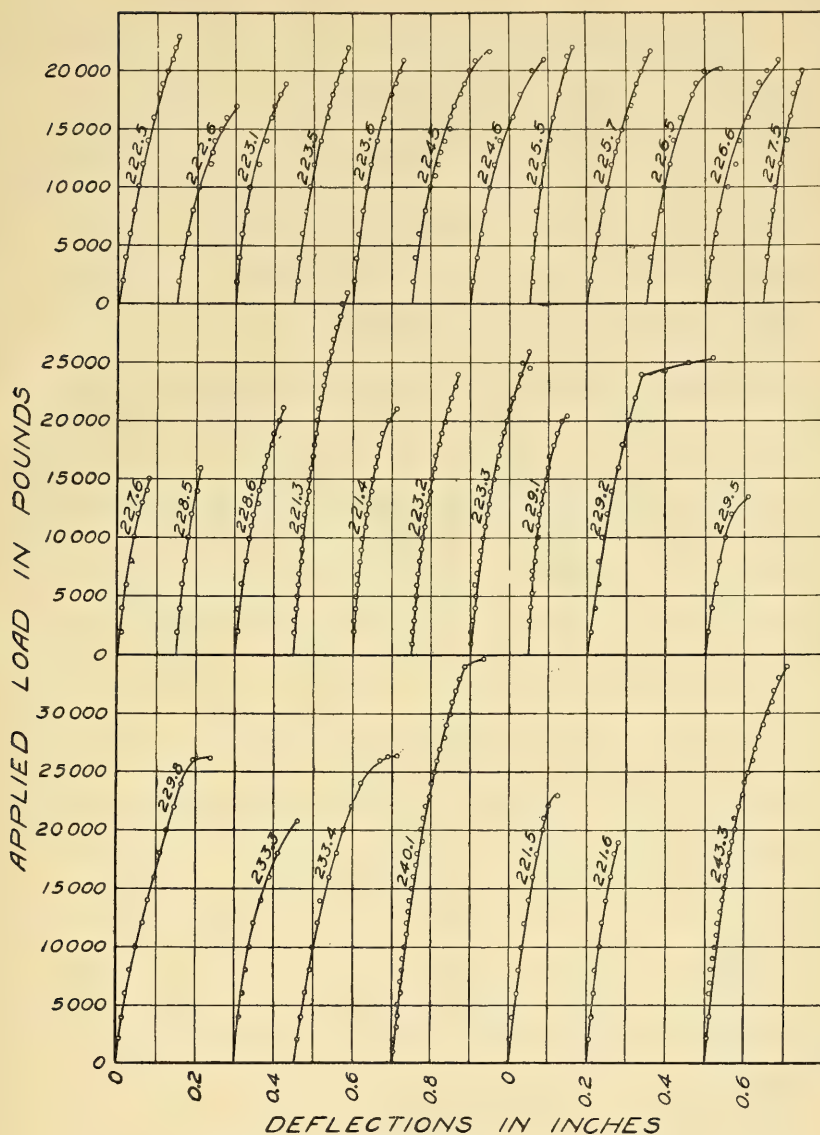


FIG. 25. LOAD-DEFLECTION DIAGRAMS.

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ON THE RATE OF FORMATION OF CARBON MONOXIDE IN GAS PRODUCERS

BY

J. K. CLEMENT



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ENGINEERING EXPERIMENT STATION

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ENGINEERING EXPERIMENT STATION

BULLETIN No. 30

FEBRUARY, 1909

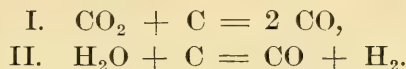
ON THE RATE OF FORMATION OF CARBON MONOXIDE IN GAS PRODUCERS

BY J. K. CLEMENT, PHYSICIST, U. S. G. S., TECHNOLOGIC BRANCH, ASSISTED BY
L. H. ADAMS, JUNIOR CHEMIST, U. S. G. S., TECHNOLOGIC BRANCH;
APPENDIX BY C. N. HASKINS, ASSISTANT PROFESSOR OF
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I. INTRODUCTORY STATEMENT.

The rapid advance in the use of producer gas in recent years has given rise to a demand for a more accurate knowledge of the processes taking place in the fuel bed of the producer and the effect on these processes of certain variations in the conditions of operation. The primary function of the gas producer is to transform solid fuel into a more readily combustible gaseous fuel. This transformation, which is relatively slow, consists of the following processes:

1. The distillation of the volatile hydro-carbons from the freshly fired fuel at relatively low temperatures.
2. The combustion of fuel by combination with the oxygen of the air.
3. The formation of producer gas proper in accordance with the equations:



The first of these reactions, the formation of carbon monoxide, is the one with which the present investigation deals. The problem proposed is in broad terms to determine the factors that govern the production of CO in the gas producer; and the effect of the temperature and of the time of contact of the gas and carbon on the percentage of CO in the producer gas. The question

of the effect of temperature was brought forth by certain experiments made by one of the writers at the Fuel Testing Plant of the U. S. Geological Survey at the Jamestown Exposition, in which it was found that the temperature in the fuel bed of the gas producer varies greatly from one portion of the bed to another. In order, therefore, to ascertain the conditions of temperature most favorable to the efficient operation of the producer, it becomes necessary to determine the temperature requisite for the formation of carbon monoxide and hydrogen in accordance with the reactions quoted in the preceding paragraph.

A study of the conditions for the reduction of CO_2 by carbon seems desirable from another consideration. A small amount of CO is invariably contained in the flue gases of boiler furnaces. It was hoped, therefore, that the investigation might furnish an explanation of the formation of CO in boiler furnaces and perhaps suggest a means of preventing such formation.

The investigations herein described were made in, and with the facilities of, the Physical Laboratory of the University of Illinois.

II. FUNDAMENTAL EQUATIONS.

According to the law of chemical mass action, a chemical reaction, as for example the reaction expressed by the equation



proceeds in one direction until equilibrium is established and then stops; and when the system is in equilibrium there is for a given temperature a certain constant relation between the amounts of the components entering into the reaction. Thus, in the system under consideration, let

[CO] = the concentration of CO in gram molecules¹ per liter,
[CO₂] = the concentration of CO₂ in gram molecules per liter;
then for equilibrium, the relation

¹A gram molecule of a substance is a weight of the substance in grams numerically equal to the molecular weight. Thus a gram molecule of CO is 28 grams, one of CO₂ is 44 grams, etc.

$$\frac{[\text{CO}]^2}{[\text{CO}_2]} = \text{constant} = K \quad (1)$$

must be satisfied.

The relation (1) may be thrown into another form as follows:

Let $100\ x =$ per cent of CO in the gas by volume;

$100\ (1 - x) =$ per cent of CO_2 in the gas by volume;

$p =$ pressure in atmospheres;

$T =$ absolute temperature;

$R =$ absolute gas constant $= 0.0821$ for the system of units here employed;

$n =$ number of gram molecules of the gas under consideration;

$v =$ volume of gas in liters.

The characteristic equation of gases is

$$pv = nRT$$

from which

$$\frac{n}{v} = \frac{p}{RT}$$

Now xn and $(1 - x)n$ are respectively the numbers of gram molecules of CO and CO_2 in the gas; hence the concentrations of CO and CO_2 are respectively

$$[\text{CO}] = \frac{xn}{v} = \frac{xp}{RT}$$

$$[\text{CO}_2] = \frac{(1 - x)n}{v} = \frac{(1 - x)p}{RT}$$

Placing these values in (1), the resulting equation is

$$\frac{x^2}{1 - x} \frac{p}{RT} = K \quad (2)$$

If the pressure and temperature are kept constant, the factor

$\frac{p}{RT}$ is a constant, and (2) may be written

$$\frac{x^2}{1 - x} = \frac{KRT}{p} = K' \quad (3)$$

where K' is a new constant.

When CO_2 gas is maintained in contact with carbon at constant temperature and pressure the two will react rapidly at first and then more slowly until the amount of CO formed is 100 x per cent of the total, where x is given by equation (3).

The relation expressed by (1) may be considered as a special case of a more general law. According to the theory of the kinetics of reactions now generally accepted in reversible reactions, two reactions take place simultaneously, one from left to right and one from right to left. In the reaction



the velocity of the reaction from left to right, that is, the rate of formation of CO, is at any instant proportional to the number of CO_2 molecules in the unit volume; thus denoting the velocity by v ,

$$v = k_1 [\text{CO}_2].$$

Similarly the velocity of the reaction from right to left, that is, the rate of formation of CO_2 , is proportional to the square of the number of CO molecules in a unit volume. Hence

$$v' = k_2 [\text{CO}]^2$$

The increase in the number of CO molecules per unit volume in the time dt is the difference of the two velocities v and v' ; that is,

$$\frac{d[\text{CO}]}{dt} = v - v' = k_1 [\text{CO}_2] - k_2 [\text{CO}]^2 \quad (4)$$

As $[\text{CO}]$ the number of gram molecules of CO in the unit volume increases and $[\text{CO}_2]$ the number of gram molecules of CO_2 decreases, the velocity $\frac{d[\text{CO}]}{dt}$ will become smaller and smaller

until finally $[\text{CO}]$ and $[\text{CO}_2]$ become constant, that is, the system attains equilibrium. In this case we have, therefore,

$$\frac{d[\text{CO}]}{dt} = v - v' = 0$$

whence

$$k_1 [\text{CO}_2] = k_2 [\text{CO}]^2. \quad (5)$$

That is, the number of CO_2 molecules formed in a given time is equal to the number that are decomposed to form CO. The last equation may be written

$$\frac{[\text{CO}]^2}{[\text{CO}_2]} = \frac{k_1}{k_2} = K. \quad (6)$$

K is called the *equilibrium constant*.

Equation (4) giving the rate of formation of CO may be modified as follows:

Let $100 a =$ per cent of CO_2 by volume at the beginning of the reaction, that is, when $t = 0$,

$100 x =$ per cent of CO by volume after the time t has elapsed.

At the beginning of the reaction, that is when $t = 0$, there is no CO, hence $x = 0$. If now n is the number of gram molecules of the gas when $t = 0$, then na is the number of gram molecules of CO_2 , and the concentration of the CO_2 is therefore

$$[\text{CO}_2] = \frac{n a}{v}.$$

But since $pv = n R T$, this relation may be written

$$[\text{CO}_2] = \frac{a p}{R T} = M \text{ gram molecules per liter.}$$

At the time t suppose m gram molecules per liter of CO to have been formed. This involves the disappearance of $\frac{m}{2}$ gram molecules per liter of CO_2 , leaving $M - \frac{m}{2}$. The number of gram molecules of gas is now

$$\begin{aligned} n (1 - a) + \left[M - \frac{m}{2} + m \right] v \\ = n (1 - a) + \left(M + \frac{m}{2} \right) \frac{n a}{M} \end{aligned}$$

$$\begin{aligned}
 &= n \left(1 + \frac{m a}{2 M} \right) \\
 &= n \left(1 + \frac{m}{2} \frac{R T}{p} \right),
 \end{aligned}$$

and the volume of the gas is therefore increased from v to $v \left(1 + \frac{m}{2} \frac{R T}{p} \right)$. The concentrations at the time t are therefore

$$[\text{CO}] = \frac{m}{1 + \frac{m}{2} \frac{R T}{p}}, \quad (7)$$

$$[\text{CO}_2] = \frac{M - \frac{m}{2}}{1 + \frac{m}{2} \frac{R T}{p}}. \quad (8)$$

But since 100 x per cent of the gas is CO by volume the concentration of the CO is also given by the relation

$$[\text{CO}] = \frac{x p}{R T}. \quad (9)$$

Combining the two expressions for $[\text{CO}]$ given by (7) and (9), we obtain.

$$m = \frac{2x}{2 - x} \frac{p}{R T},$$

and introducing this expression for m in (8), the result after slight reduction is

$$[\text{CO}_2] = \left(a - \frac{a + 1}{2} x \right) \frac{p}{R T}. \quad (10)$$

Introducing in equation (4) the expressions for the concentrations given by (9) and (10), the result is

$$\frac{d[\text{CO}]}{dt} = k_1 \frac{p}{R T} \left(a - \frac{a + 1}{2} x \right) - k_2 \left[\frac{p}{R T} \right]^2 x^2. \quad (11)$$

But from (9)

$$\frac{d[\text{CO}]}{dt} = \frac{p}{R T} \frac{dx}{dt},$$

whence

$$\frac{dx}{dt} = k_1 \left(a - \frac{a+1}{2} x \right) - k_2 \frac{p}{RT} x^2.$$

Replacing the constant $k_2 \frac{p}{RT}$ by a single symbol k'_2 , the final equation for the reaction velocity is

$$\frac{dx}{dt} = k_1 \left(a - \frac{a+1}{2} x \right) - k'_2 x^2. \quad (12)$$

The integration of the differential equation (12) offers no great difficulty.

The determination of the constants k_1 and k_2 seems to have been made hitherto by assuming the value of the ratio $\frac{k_1}{k'_2}$. This

method is applicable when the equilibrium conditions are readily realized. As, however, this is not the case in the present reaction it has been necessary to devise a method for the determination of k_1 and k'_2 from two or more pairs of simultaneous observations of x and t . It turns out that the method is applicable not only to the reaction in question, but also to the most general incomplete reactions of the second order.

The great difficulty of realizing, with certainty, the condition of the equilibrium in reactions like the one under consideration, makes it highly desirable, therefore, to obtain a general solution of the differential equation, without introducing a particular numerical

value for the ratio $\frac{k_1}{k'_2}$. We are indebted to Prof. C. N.

Haskins for the following solution, the developments of which will be found in the appendix:

$$x = \frac{4a}{a+1} \frac{\gamma \tanh at}{1 + \gamma \tanh at}. \quad (13)$$

In equation (13) a and γ are determined by the relation

$$k_1 = \frac{4a\gamma}{a+1},$$

$$k'_2 = \frac{a(a+1)}{4a\gamma} (1 - \gamma^2).$$

When the initial percentage of CO_2 is 100, then $a = 1$,

$$\frac{dx}{dt} = k_1 (1 - x) - k'_2 x^2,$$

and

$$x = \frac{2\gamma \tanh at}{1 + \gamma \tanh at}. \quad (14)$$

In the case of the gas producer the value of a is about 0.21.

III. BOUDOUARD'S EXPERIMENTS.

An elaborate series of determinations of the amount of CO formed at different temperatures has been made by O. Boudouard.¹ Boudouard's observations were made at 650° , 800° and 925° C. In his experiments at 650° and 800° glass tubes containing charcoal, coke, retort carbon or lamp black were filled with CO_2 , heated to the temperature of the experiment and then sealed. The tubes were maintained at constant temperature until equilibrium was reached, that is, when further heating at the same temperature produced no increase in the percentage of CO present. At 650° the heating was continued for twelve hours before equilibrium was attained. At 800° equilibrium was reached in one hour in the tubes containing charcoal and in two and one-half hours in those containing lamp black. With coke and retort carbon the process was not complete at the end of nine hours. In the experiment at 925° the carbon was heated in a porcelain tube, through which was passed a stream of CO_2 gas. The average time of contact between CO_2 and carbon calculated from the data given

¹O. Boudouard, *Comptes Rendus de l'Academie des Sciences*,

Vol. 128, page 824, 154; 1899.

Vol. 131, page 1204, 1900.

Vol. 130, page 132, 1900.

Bulletin Soc. Chim., Paris, Vol. 21, 1901.

Bulletin Soc. Chim., Paris, Vol. 25, 1901.

Ann. de Chimie et de Physique, Vol. 354, 1901.

See also Haber, *Thermodynamics of Technical Gas Reactions*, 1908, p. 311.

in Boudouard's account of his experiments was approximately 30 seconds.

A summary of Boudouard's results is given in the following table:

TEMPERATURE C. ^o	PER CENT OF CO ₂	PER CENT OF CO
650.	61.	39.
800.	7.	93.
925.	4.	96.

These values have been made the basis of computation by many writers on the chemistry of combustion and of the water gas reaction and especially in treatises on the gas producer.

In the first references to Boudouard's work which came to the attention of the writers, no notice was taken of the remarkably low reaction velocity of the formation of CO from CO₂ and carbon, and of the great length of time required to obtain the percentages of CO that are given in the preceding table. In at least one case the values shown above were offered as representing the quality of gas that should be obtained in the gas producer at the temperatures given. The writers were therefore led to regard Boudouard's figures as defining the relative proportions of CO₂ and CO that should be formed in a gas producer at various temperatures of the fuel bed.

The experiments which form the subject of this paper had originally as their object the confirmation of Boudouard's results as well as a continuation of them at higher temperatures. Preliminary experiments made by Dr. C. S. Hudson demonstrated that the amount of CO formed at a given temperature depends largely on the time of contact or in other words on the rate of flow of the gas through the fuel bed. Apparently Boudouard's results represent limiting values, which can be obtained only with a very low gas velocity. In order to ascertain the conditions for the formation of CO in producer furnaces, it is necessary, therefore, to determine the rate of formation of CO from CO₂ and carbon at various temperatures; that is, to determine the

amount of CO formed with different rates of flow of the gas through the fuel bed.

IV. METHOD OF EXPERIMENT.

The arrangement of the apparatus is shown in Fig. 1, 2, and 3. A porcelain tube of 1.5 cm inside diameter, 60 cm long, and glazed on the outside was filled with charcoal, coal, or coke and heated in an electric furnace. The furnace, which was designed

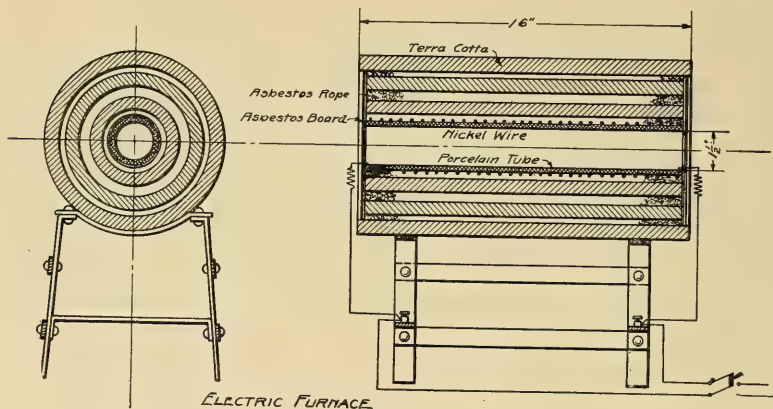


FIG. 1

especially for this investigation, is shown in detail in Fig. 3. It has been operated continuously for a period of six months at temperatures of from 800° to 1200° , and even 1300° C. The temperature inside the porcelain tube could be maintained at any value up to 1300° C with no fluctuations greater than 1° or 2° . The



FIG. 2

heating coil consists of a coil of No. 13 nickel wire, wound with eight turns per inch on an electrical porcelain insulating tube of 38 mm inside diameter and 35 cm long. At either end of the coil

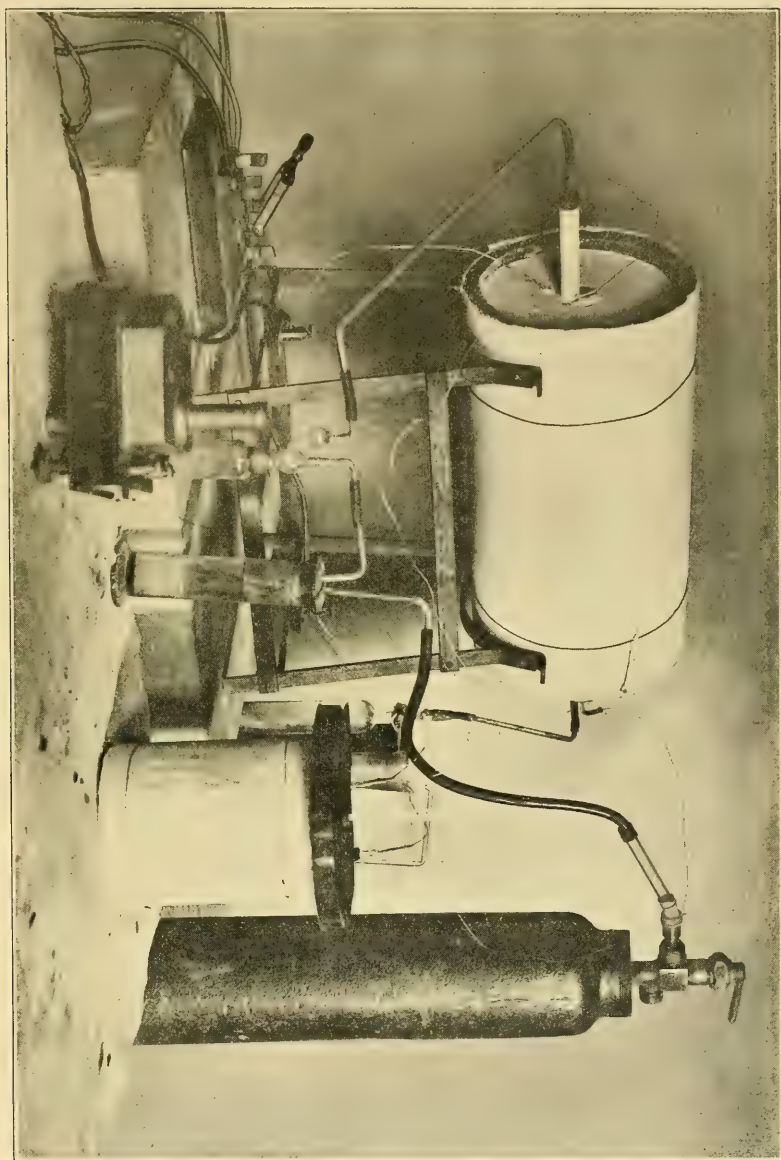


FIG. 3

the number of turns was increased slightly to compensate for the cooling effect of the end at the furnace. As a protection against corrosion, the coil was painted over with a thin layer of magnesite cement—a material that is capable of withstanding very high temperatures. The heating tube was then mounted inside of and concentric with several terra cotta pipes and the spaces between the pipes were filled with light calcined magnesia. The cost of the material used in the furnace and the amount of labor required were very small. A temperature of 1000°C could be maintained by the expenditure of 600 watts.

The temperature inside the porcelain tube was measured by means of a platinum platinum-rhodium thermocouple (see Fig. 2) and a Siemens & Halske milli-voltmeter. As the thermo-electric height of the couple fell slightly with use at high temperature—due probably to the reducing action of CO gas on the insulating tubes and the consequent contamination of the couple—it was found necessary to calibrate the couple from time to time. This was accomplished by determination of the melting points of zinc, silver and copper. The error of individual temperature observations does not exceed 5° below 1100° and 10° to 15° between 1100° and 1300° .

The carbon with which the porcelain tube was filled was crushed to pieces of a uniform size—about 5 mm on a side. Only the central portion of the tube (see Fig. 2) contained carbon, the remainder of the space being occupied by pieces of broken porcelain, which served at one end to heat the gas entering the tube, and at the other end, by reducing the size of the passage way, to increase the velocity of the gas through the region of falling temperature. Through the porcelain tube was passed a stream of CO_2 gas. In the earlier experiments CO_2 was prepared from marble and hydrochloric acid. Later, CO_2 was taken from a tank of liquid carbon dioxide.

The velocity of the gas over the carbon was determined by the dimensions of the tube, the weight and density of the carbon,

and the temperature and the volume of gas passed through the tube per minute.¹

The analyses were made by the Hempel method, both CO₂ and CO being absorbed. The amount of gas remaining in the burette after the absorption in cuprous chloride was seldom greater than two per cent.

V. EXPERIMENTS WITH CHARCOAL.

With the apparatus described in the preceding pages, experiments were conducted at temperatures ranging from 700° to 1300° C. The experiments with charcoal extended over a period of several months. The results are contained in tables 1-6.

TABLE 1
RATE OF FORMATION OF CO FROM CO₂ AND CHARCOAL
AT A TEMPERATURE OF 800° C.

$$k_1 = 0.01968$$

$$k_2 = 3.031$$

Time of contact in seconds <i>t</i>	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
∞	0		0.535
188.6	0.0053	0.503	0.534
115.9	0.0086	0.504	0.527
57.18	0.0175	0.518	0.508
45.70	0.0219	0.522	0.468
24.20	0.0413	0.375	0.345
15.50	0.0645	0.283	0.252
12.32	0.0810	0.245	0.209
2.686	0.354	0.063	0.051
1.550	0.645	0.039	0.030

¹The increase in the volume of the gas in its passage through the reaction tube, due to the formation of two CO molecules in place of every molecule of CO₂ which disappears, makes it difficult to determine accurately the time of contact and consequently the velocity of the gas.

The values of *t*, the time of contact, given in the following tables are based on the volume of gas leaving the tube and are therefore somewhat too low. Since the major portion of the expansion takes place within a short distance from the entrance to the tube, the error here introduced is probably not appreciable.

TABLE 2

RATE OF FORMATION OF CO FROM CO₂ AND CHARCOAL

AT A TEMPERATURE OF 850° C.

$$k_1 = 0.07174$$

$$k_2 = 3.238$$

Time of contact in seconds t	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
∞	0		0.742
123.0	0.0082	0.743	0.742
54.18	0.0184	0.702	0.741
24.43	0.0410	0.572	0.694
13.23	0.0756	0.526	0.564
9.268	0.1070	0.297	0.463
4.630	0.216	0.297	0.281
3.684	0.271	0.224	0.231
3.254	0.307	0.225	0.207

TABLE 3

RATE OF FORMATION OF CO FROM CO₂ AND CHARCOAL

AT A TEMPERATURE OF 900° C.

$$k_1 = 0.1540$$

$$k_2 = 2.599$$

Time of contact in seconds t	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
∞	0		0.873
64.29	0.0156	0.873	0.873
41.18	0.0226	0.867	0.872
10.008	0.0999	0.708	0.739
4.257	0.234	0.498	0.472
2.840	0.352	0.311	0.351
2.172	0.461	0.344	0.284

TABLE 4

RATE OF FORMATION OF CO FROM CO₂ AND CHARCOAL

AT A TEMPERATURE OF 925° C.

$$k_1 = 0.2175$$

$$k_2 = 2.298$$

Time of contact in seconds t	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
∞	0		0.914
118.8	0.0084	0.947	0.914
81.2	0.0123	0.933	0.914
12.37	0.0807	0.848	0.875
5.80	0.1725	0.718	0.697
4.277	0.234	0.642	0.595
2.272	0.440	0.375	0.387

TABLE 5

RATE OF FORMATION OF CO FROM CO₂ AND CHARCOAL

AT A TEMPERATURE OF 1000° C.

$$k_1 = 0.6404$$

$$k_2 = 4.708$$

Time of contact in seconds t	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
∞	0		0.942
70.0	0.0143	0.949	0.942
18.60	0.0538	0.943	0.941
8.245	0.1195	0.903	0.938
3.675	0.272	0.797	0.869
2.296	0.436	0.795	0.752

TABLE 6
RATE OF FORMATION OF CO FROM CO₂ AND CHARCOAL
AT A TEMPERATURE OF 1100° C.

$$k_1 = 1.495$$

$$k_2 = 5.275$$

Time of contact in seconds t	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
∞	0		0.972
36.48	0.0274	0.987	0.972
10.43	0.0958	0.983	0.972
4.968	0.2010	0.981	0.971
3.640	0.2745	0.973	0.968
1.921	0.521	0.946	0.955

The first and second columns of each table give the time of contact t and the reciprocal of the time of contact $\frac{1}{t}$, which is equal to the velocity of the gas divided by the length of the charcoal column; thus

$$\frac{1}{t} = \frac{v}{l}.$$

The third column contains the percentages of CO observed, and the values in the last column were calculated by means of equation (13), viz:

$$x = \frac{4a}{a+1} \frac{\gamma \tanh at}{1 + \gamma \tanh at}$$

The method of computing a and γ of this equation is described in the appendix. The constants k_1 and k'_2 are determined by the relations,

$$k_1 = \frac{4a\gamma}{a+1}$$

$$k'_2 = \frac{a(a+1)}{4a\gamma} (1 - \gamma^2) \quad .$$

Values of k_1 , k_2 , k'_2 , α , and γ are given in table 7.

TABLE 7
CONSTANTS USED IN COMPUTATION OF x

Temp. C°	α ($\alpha = 1$)	γ ($\alpha = 1$)	k'_2	k_2	k_1	$K = \frac{k_1}{k_2}$
800.	0.0276	0.3568	0.03373	3.031	0.01968	0.006493
850.	0.0612	.5853	0.03443	3.238	0.07174	0.02216
900.	0.09998	.7711	0.02646	2.599	0.1540	0.05925
925	0.1297	.8388	0.02291	2.298	0.2175	0.09465
1000	0.3617	.8853	0.04416	4.708	0.6404	0.13603
1100	0.7921	.9437	0.04588	5.275	1.4950	0.28341

The calculated and observed values of x , the per cent of CO, agree within two or three per cent.

A comparison of the results in tables 1 to 6 shows in the first place that with increasing temperature there is a rapid increase in the percentage of CO obtained with any given rate of

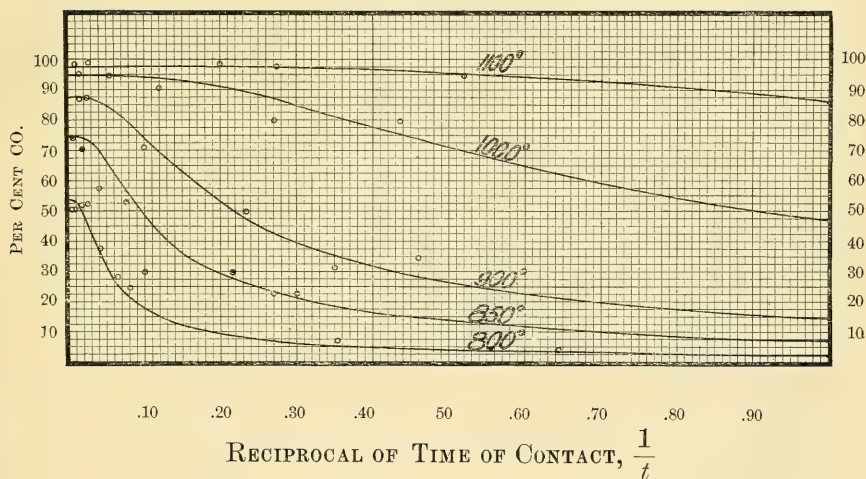


FIG. 4.

flow of the gas; in the second place that with increasing gas velocity at low temperatures the percentage of CO formed falls off very rapidly, at higher temperatures very slowly. These variations are illustrated by the curves in Fig. 4 in which the per-

centage of CO is plotted as a function of $\frac{1}{t} = \frac{v}{l}$. When l , the

length of the charcoal column, is equal to one, i. e., is equal to the unit of length, then the numbers along the abscissa give the velocity of the gas in terms of the same unit of length, and per second. For example, the length of the charcoal column in the experiments here recorded, was approximately 20 cm. The velocity

corresponding to the point $\frac{1}{t} = 1$ at the extreme right of Fig. 4, is therefore 20 cm. per second.

The general shape of all the curves in Fig. 4 is the same. The percentage of CO is greatest at zero velocity. With

increasing values of $\frac{1}{t}$ each curve falls off, slowly at first

then more rapidly, passing a point of inflection and finally becoming nearly horizontal. The intersections of the curves with the CO axis give the percentage of CO corresponding to the condition of equilibrium.

That a considerable amount of time is required to reach equilibrium in the reaction under consideration, is further illustrated in Fig. 5, in which the percentage of CO is plotted as a function of t , the time of contact. (One small division = 1 second). At 800° for example, the percentage of CO reaches a practically constant value at the end of 50 sec.; at 1000° in 6 sec.

The curves in Fig. 4 were plotted from values of x (= per cent CO) calculated from equation (13). The observed values are indicated by the small circles. By means of equation (13) it is possible to calculate the per cent of CO corresponding to any given gas velocity providing a and γ or k_1 and k_2 are known.

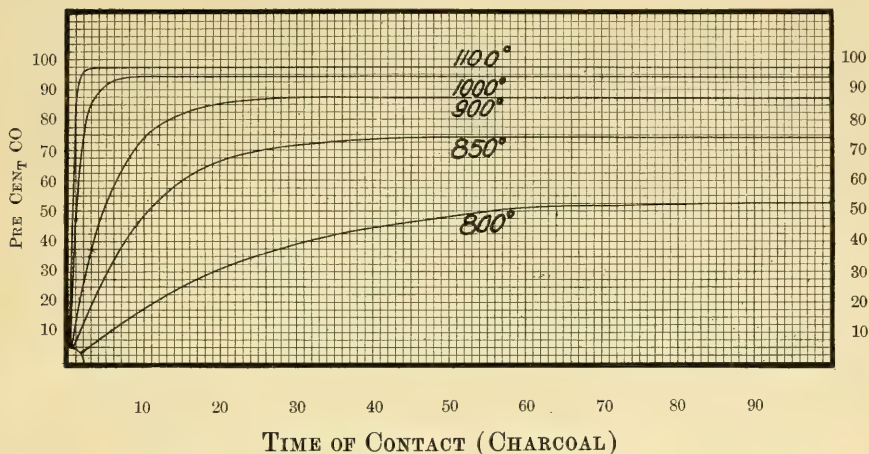


FIG. 5.

VI. VARIATIONS OF K AND k_1 WITH TEMPERATURE.

The values of k_1 and K given in table 7 exhibit a systematic variation with temperature. If equations can be found that will express k_1 and K as functions of the temperature it will then be possible to calculate the per cent of CO for any time of contact and any desired temperature. Such equations have been deduced by Van't Hoff from purely thermodynamical considerations. They are the following.¹

$$\frac{d(\ln K)}{dT} = - \frac{Q}{R T^2}, \quad (15)$$

and

$$\frac{d(\ln k_1)}{dT} = \frac{A}{T^2} + B. \quad (16)$$

In these equations, Q is the latent heat of reaction at the absolute temperature T , A is a function of Q but is selected arbitrarily, and B is an arbitrary function of the temperature. By integration the latter equation becomes

¹The symbol $\ln$ in the following equations stands for natural logarithm.

$$\ln k_1 = -\frac{A}{T} + B T + C, \quad (17)$$

where C is an integration constant. The values of A , B , and C , in equation (5) have been determined from the simultaneous values of k_1 and T of tables 1-6. Table 8 contains the values of k_1 , obtained at various temperatures as well as the values of k_1 calculated from equation (17). The agreement is remarkable good.

TABLE 8

VARIAION OF k_1 WITH TEMPERATURE (CHARCOAL)

$$\ln k_1 = -\frac{50910}{T} - 0.0203 T + 65.376$$

Temp. Deg. C.	Absolute Temp. T	k_1 (obs)	k_1 (calc)
800	1073	0.020	0.021
850	1123	0.073	0.064
900	1173	0.154	0.159
925	1198	0.217	0.237
1000	1273	0.640	0.629
1100	1373	1.490	1.53

In order to integrate the equation

$$\frac{d(\ln K)}{dT} = -\frac{Q}{RT^2}$$

it is first necessary to determine the heat Q as a function of the temperature. It has been shown by Kirchoff that the increase of Q per degree rise in temperature is equal to the difference of the molecular heats of the factors and of the products of the reaction. Following this law, and taking the specific heat of a factor or product as a linear function of the temperature, which is very nearly true for gases, the relation between Q and T is given by the equation¹

$$Q = Q_0 + c_1 T + c_2 T^2 \quad (18)$$

In this equation Q_0 denotes the heat of reaction for $T = 0$, and c_1 and c_2 are obtained as follows: Assuming that the mean specific heat of each gas is given by an expression of the form

¹Haber, *Thermodynamics of Technical Gas Reactions*, p. 49, eq. (7a).

$$c = a + b T,$$

then c_1 is the difference between the sum of the a 's of the factors and the sum of the a 's of the products; likewise c_2 is the sum of the b 's of the factors less the sum of the b 's for the products. Substituting the value of Q given by (18) in (15) the result is

$$\frac{d(\ln K)}{dt} = -\frac{1}{R} \left(\frac{Q_0}{T^2} + \frac{c_1}{T} + c_2 \right)$$

whence by integration

$$\ln K = \frac{1}{R} \left(\frac{Q_0}{T} - c_1 \ln T - c_2 T \right) + C. \quad (19)$$

The constants in this equation, (19), may be determined by either of two different methods: by experimental determinations of Q_0 , c_1 , and c_2 , or from four or more simultaneous observations of K_{obs} and T . The first method was adopted in this instance, the following being the values of the quantities in question:

$$\begin{aligned} Q_0 &= -40166 \\ c_1 &= -2.055 \\ c_2 &= 0.003104 \\ C &= 8.604 \end{aligned}$$

In the determination of c_1 and c_2 , Langen's values for the specific heats of CO and CO₂ and the value of Kunz for the specific heat of charcoal were employed.

The value of R (in gram-calories per deg.) is 1.985. Hence taking the above constants and this value of R , (19) reduces to

$$\ln K = -\frac{20235}{T} + 1.035 \ln T - 0.001564 T + 8.604. \quad (20)$$

Table 9 gives values of K calculated from equation (20) along with values (marked K_{obs}) obtained from observed values of x and T in tables 1-6. In the fourth column are the observed values of x , the amount of CO in equilibrium with CO₂ and charcoal at temperatures from 800° to 1100°; and in the fifth columns the values of x corresponding to the values of K in the third column.

The constants of equation (19) were calculated also by the method of least squares, from simultaneous values of K_{obs} and T , but the agreement was less satisfactory than by the first method.

TABLE 9
VALUES OF K AND OF x_{∞}

Temp. C.	K (obs)	K (cal)	x_{∞} (obs)	x_{∞} (cal)	x_{∞} (obs) Boudouard
500		0.000007		0.021	
600		0.00013		0.093	
650		0.00046		0.185	0.39
700		0.00137		0.283	
800	0.0065	0.0090	0.526	0.582	0.93
850	0.022	0.020	0.738	0.722	
900	0.059	0.042	0.871	0.832	
925	0.094	0.060	0.912	0.873	0.96
1000	0.136	0.151	0.939	0.945	
1100	0.283	0.448	0.971	0.981	
1200		1.120		0.994	
1300		2.455		0.997	
1400		4.826		0.9985	
1500		8.671		0.9992	
1600		14.44		0.9996	

The agreement between "observed" and "calculated" values of K and k_1 in tables 8 and 9 shows that the changes of K and k_1 , with temperature follow van't Hoff's laws. It is possible, therefore, by means of equations (17) and (19) and the values of the constants of these equations given in tables 8 and 9, to compute K and k_1 for any desired temperatures. Having the values of K and k_1 and consequently of k_2 ($k_2 = \frac{k_1}{K}$) the per cent x of CO corresponding to any time of contact t can then be calculated by means of equation (13).

VII. EXPERIMENTS AT 700°.

In Fig. 4 the curve for 800° falls off very rapidly with increasing rate of flow of gas. At this temperature the gas velocity must be exceedingly low to obtain the equilibrium percentage of CO.

At temperatures below 800° it was practically impossible to reach equilibrium with a finite gas velocity. A great number of experiments were made at 700° but the results were too inconsistent to admit of mathematical treatment. Some of the observations are given in the following table:

TABLE 10
OBSERVATIONS AT 700°C. (CHARCOAL).

t = Time of contact in seconds	$\frac{1}{t}$	% CO/100
86.9	0.0115	0.012
86.2	0.0116	0.155
99.9	0.0100	0.077
23.4	0.0426	0.004
15.0	0.0668	0.014
7.11	0.1405	0.009
9.71	0.103	0.022
5.60	0.178	0.006
5.02	0.199	0.008
4.18	0.239	0.012

These results show that, except at exceedingly low velocities, the amount of CO formed was never greater than one or two per cent.

VIII. EXPERIMENTS WITH COKE AND COAL.

The experiments with coke and coal were conducted in the same manner as with charcoal. The material was crushed to pieces about 5 mm on a side. The constants α and γ of equation (13) were obtained for each temperature by the method given in the appendix. Tables 11-15 contain the results of the observations with coke. In the last column of each table are given the values of x , the percentage of CO formed, calculated from equation (13).

TABLE 11

RATE OF FORMATION OF CO FROM CO₂ AND COKE

AT A TEMPERATURE OF 900° C.

$$k_1 = 0.00231$$

$$k_2 = 0.03686$$

Time of contact in seconds. t	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
142.0	0.0070	0.276	0.278
80.20	0.0124	0.131	0.169
43.91	0.0228	0.094	0.096
24.82	0.0403	0.057	0.056
16.11	0.0620	0.049	0.037
9.575	0.1045	0.026	0.023
3.741	0.2671	0.008	0.009

TABLE 12

RATE OF FORMATION OF CO FROM CO₂ AND COKE

AT A TEMPERATURE OF 1000° C.

$$k_1 = 0.02323$$

$$k_2 = 0.3591$$

Time of contact in seconds t	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
123.2	0.0081	0.784	0.866
80.25	0.0125	0.644	0.795
33.25	0.0301	0.529	0.527
18.72	0.0535	0.320	0.350
6.37	0.1571	0.139	0.138
4.101	0.2439	0.115	0.091
3.072	0.3258	0.092	0.069
1.983	0.5045	0.063	0.045

TABLE 13

RATE OF FORMATION OF CO FROM CO₂ AND COKE

AT A TEMPERATURE OF 1100° C.

$$k_1 = 0.1335$$

$$k_2 = 0.5296$$

Time of contact in seconds t	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
90.00	0.0111	0.971	0.971
29.92	0.0334	0.854	0.955
13.20	0.0758	0.661	0.817
6.765	0.1476	0.556	0.592
3.198	0.3135	0.317	0.346
1.784	0.5606	0.304	0.211
1.660	0.6030	0.240	0.1942
1.590	0.6299	0.221	0.190
1.462	0.6840	0.214	0.177
0.962	1.0399	0.133	0.121

TABLE 14

RATE OF FORMATION OF CO FROM CO₂ AND COKE

AT A TEMPERATURE OF 1200° C.

$$k_1 = 0.4095$$

$$k_2 = 0.6718$$

Time of contact in seconds t	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
18.92	0.0528	0.989	0.987
12.70	0.0788	0.978	0.983
8.250	0.1213	0.953	0.956
2.402	0.4160	0.685	0.624
1.582	0.6320	0.439	0.460
1.080	0.9260	0.335	0.357

TABLE 15

RATE OF FORMATION OF CO FROM CO₂ AND COKE

AT A TEMPERATURE OF 1300° C.

$$k_1 = 1.483$$

$$k_2 = 0.7313$$

Time of contact in seconds <i>t</i>	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
8.860	0.1129	0.999	0.997
4.149	0.2415	0.979	0.997
2.100	0.4760	0.932	0.955
1.130	0.8850	0.834	0.816

The results with coke are shown graphically in Fig. 6. The curves for 900°, 1000° and 1100° are considerably lower than the curves with charcoal for the same temperatures, except for very low velocities.

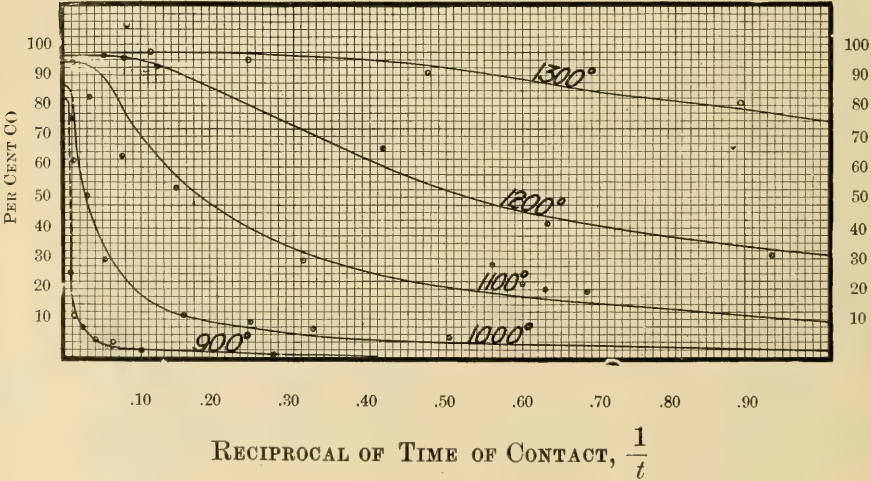


FIG. 6.

TABLE 16

RATE OF FORMATION OF CO FROM CO₂ AND ANTHRACITE COAL

AT A TEMPERATURE OF 1100° C.

$$k_1 = 0.119$$

$$k_2 = 1.410$$

Time of contact in seconds t	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
34.20	0.0293	0.8780	0.912
9.370	0.1069	0.6010	0.657
5.415	0.1848	0.4770	0.472
3.301	0.3026	0.3020	0.322
2.439	0.4101	0.2650	0.251

TABLE 17

RATE OF FORMATION OF CO FROM CO₂ AND ANTHRACITE COAL

AT A TEMPERATURE OF 1200° C.

$$k_1 = 0.2374$$

$$k_2 = 0.1767$$

Time of contact in seconds t	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
47.05	0.0212	0.997	0.993
10.39	0.0964	0.856	0.901
5.070	0.1971	0.715	0.688
2.845	0.3516	0.423	0.472
1.592	0.6270	0.310	0.309

TABLE 18

RATE OF FORMATION OF CO FROM CO₂ AND ANTHRACITE COAL

AT A TEMPERATURE OF 1300° C.

$k_1 = 0.5791$

$k_2 = 0.2016$

Time of contact in seconds <i>t</i>	$\frac{1}{t}$	% CO/100 Observed	% CO/100 Calculated
12.40	0.0806	0.999	0.997
6.030	0.1659	0.965	0.968
3.600	0.2779	0.824	0.876
2.980	0.3358	0.809	0.822
1.908	0.5249	0.663	0.668
1.070	0.9350	0.503	0.462

The observations with anthracite coal are given in tables 16, 17, and 18, and are illustrated graphically in Fig. 7.

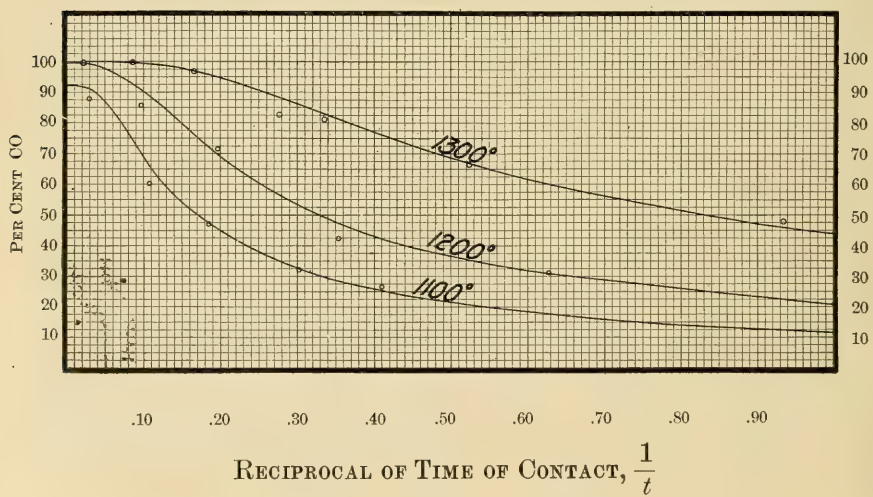


FIG. 7.

Here the curves fall off even more rapidly than the curves for coke in Fig. 6. With very low velocities, that is, when the time

of contact is sufficient for the reaction to reach equilibrium, the percentage of CO formed is practically the same with each of the three forms of carbon. As the rate of flow of the gas increases, the effect of the difference in the reaction velocities becomes more appreciable.

TABLE 19
VALUES OF x_{∞} FOR CHARCOAL, COKE, AND COAL.

Temp. C.	x_{∞} (cal)	x_{∞} (obs) Charcoal	x_{∞} (obs) Coke	x_{∞} (obs) Coal
900	0.832	0.871	0.875	
1000	0.945	0.939	0.886	
1100	0.981	0.971	0.968	0.914
1200	0.994		0.987	0.994
1300	0.997		0.996	0.997

Values of x_{∞} the percentages of CO in equilibrium with CO₂, and charcoal, coke, and coal respectively, are given in table 19. The values in the second column of this table were calculated from the values of K in table 9, by means of the equation

$$\frac{x^2}{1-x} = K \frac{RT}{p},$$

A comparison of Fig. 5, 6, and 7 shows that the reaction velocity is greatest with charcoal and lowest with anthracite coal. The temperature coefficient of k_1 , the coefficient of reaction velocity, was determined for coal and coke in the same manner as for charcoal. The "observed" and "calculated" values of k_1 are shown in tables 20 and 21.

The constant k_2 in the equation

$$\frac{d[\text{CO}]}{dt} = k_1 [\text{CO}_2] - k_2 [\text{CO}]^2$$

is the coefficient of reaction velocity of the reaction



taken from right to left. At the temperatures of these experiments, 800°-1300°, the carbon produced by the decomposition of

CO is in the form of lamp black, regardless of the form of carbon present in the reaction tube, viz: charcoal, coke, or coal. At any one temperature, therefore, k_2 should be the same in all three cases. From a comparison of tables 1-6, 11-15 and 16-18 it will be seen that there is considerable deviation in the values of k_2 for the three forms of carbon used. This is doubtless due in part to experimental errors. There is a further consideration, however, to which attention should be called, viz: that the reaction in question is not reversible. The lamp black produced by the reverse reaction



is not identical physically with the form of carbon, charcoal, or coke that is consumed in the formation of CO. Consequently the law of chemical mass action is not strictly applicable. In the systems under consideration, equilibrium would not be reached until all the carbon has been transformed to lamp black.

TABLE 20

VARIATION OF k_1 WITH TEMPERATURE (COKE)

$$\ln k_1 = -\frac{47220}{T} - 0.009699 T + 45.597$$

Temp. C.	T	k_1 (obs)	k_1 (calc)
900	1173	0.0024	0.0023
1000	1273	0.021	0.023
1100	1373	0.121	0.134
1200	1473	0.473	0.410
1300	1573	1.38	1.48

TABLE 21

VARIATION OF k_1 WITH TEMPERATURE (ANTHRACITE COAL)

$$\ln k_1 = \frac{31972}{T} + 0.02272 T - 56.607$$

Temp. C.	T	k_1
1100	1373	0.119
1200	1473	0.237
1300	1573	0.579

IX. APPLICATION OF EXPERIMENTAL RESULTS TO THE PROCESSES OF THE GAS PRODUCER AND BOILER FURNACE.

As stated in the introduction, the experiments here described were undertaken primarily to determine the temperature necessary for the formation of high percentage CO gas in the fuel bed of the gas producer, and to ascertain the conditions that govern the formation of CO in boiler furnaces. The results here presented indicate that the amount of CO formed in the gas producer depends on three factors: (1) the temperature; (2) the depth of the hot portion of the bed; and (3) the rate of flow of gas through the bed. Stated in a more concise form, the percentage of CO formed depends on the temperature and the time of contact of gas and carbon, i. e., the average time required for a molecule of gas to pass through the fuel bed. The variation of the percentage of CO with the rate of flow of gas is illustrated in Fig. 4, 6 and 7. The curves for coke, Fig. 6, may be taken as representing the conditions in the fuel bed of the producer. At 1300° C., for example, with zero velocity (time of contact = ∞)

practically all the CO₂ will be converted to CO; when $\frac{1}{t} = 0.5$,

(time of contact $t = 2$ sec.), 90 per cent CO is obtained; and when $t = 1$ only 80 per cent CO is formed. In a fuel bed one foot in depth, since

$$\frac{1}{t} = \frac{\text{velocity of gas}}{\text{depth of fuel bed}} = \frac{v}{l}$$

a time of contact of $t = 2$ sec. corresponds to a velocity 0.5 ft. per sec. and $t = 1$ to a velocity of 1 ft. per sec. At 1300° C., then, in a fuel bed one foot in depth, with a velocity of 0.5 ft. per sec. 90 per cent of CO would be formed and with a velocity of 1 ft. per sec. 80 per cent. In a fuel bed two feet in depth, the gas velocities corresponding to the same percentage of CO would be twice as great. In other words, for given conditions of temperature and quality of gas, the depth of bed and velocity of gas

must vary proportionally and their ratio $\frac{v}{l}$ must remain constant. A fuel bed one foot in depth and a gas velocity of one foot per second should yield the same percentage of CO as a bed two feet in depth with a gas velocity of 2 ft. per sec.

It is impossible to determine accurately the velocity of the gas through the producer fuel bed, on account of the difficulty of estimating the magnitude of the passages through the bed.¹ The velocity lies probably between 0.5 and 5.0 ft. per sec. The right half of the curves in Fig. 6 lies within these limits and therefore corresponds approximately to the conditions of producer operation.

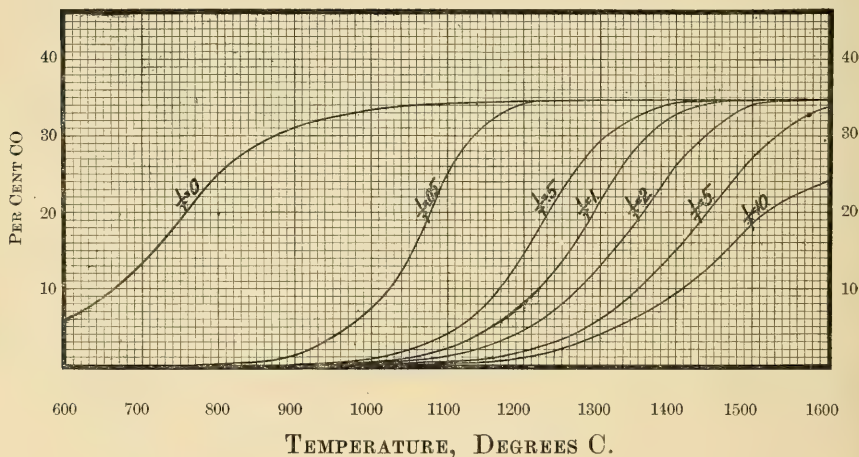


FIG. 8.

In Fig. 8 is shown graphically the variation with temperature of the amount of CO formed with different values of $\frac{1}{t}$. The

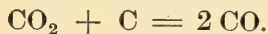
¹When any given number of pounds of air is passed per second through a fuel bed of given dimensions, the velocity through the bed will increase as the percentage of voids is decreased. Thus the velocity will be much higher with slack coal than with uniformly sized nut coal. Further, the per cent of voids will be influenced by the amount of coking and clinkering.

ordinate is the per cent of CO in gas containing initially 21 per cent CO_2 (air in which the oxygen has been converted quantitatively to CO_2). The abscissa is temperature in degrees Centigrade.

The upper curve, $\frac{1}{t} = 0$, represents the maximum amount of CO which could be produced from air. The intersection of the curve for any velocity with a given horizontal line, for example, the line for CO = 30 per cent, gives the temperature required to form that amount of CO with the particular velocity. Thus to obtain 30 per cent CO with a velocity of one foot per sec. (length of bed = 1 ft.) will require a temperature of 1360°C ., and with a velocity of 2 ft. per sec., 1435° . The curves of Fig. 6 and 8 indicate that the temperature of the producer bed should not be less than 1300°C .

These investigations demonstrate that a very high temperature is necessary for the production of CO from CO_2 and carbon. There are other considerations, however, which are opposed to the operation of the fuel bed of the gas producer at extremely high temperature—above 1300°C .: A high temperature of fuel bed means that the gases will leave the producer at a high temperature and thus lower the efficiency of the producer. The gain in capacity will therefore be accompanied by a loss in efficiency, unless the heat of the gases can be used efficiently for generating steam and preheating the air blast. Also a high temperature favors clinkering. In the application of the results of these experiments to commercial producers and furnaces it will be necessary of course to consider the various questions that are involved.

Various explanations have been suggested to account for the presence of small amounts of carbon monoxide in the flue gases of boiler furnaces. Perhaps the one most generally accepted by engineers is that the oxygen of the air first unites with carbon to form CO_2 and that as this gas passes up through the hot fuel bed it combines with carbon in accordance with the equation



Assuming this to be the correct explanation, then the question to be solved is what conditions are favorable to this reaction and what conditions will tend to retard it. In the preceding paragraphs it has been shown that the higher the velocity of the gas and thinner the fuel bed, the less will be the percentage of CO formed. A heavy fuel bed in the boiler furnace would therefore favor the formation of CO. Also, the greater the supply of air to a given depth of bed, the less should be the tendency to form CO.

X SUMMARY AND CONCLUSIONS.

1. The rate of formation of CO in the reaction,



has been determined with charcoal from 800° to 1100° C., with coke from 900° to 1300° C., and with anthracite coal from 1100° to 1300° C.

2. The differential equation for the velocity of incomplete reactions

$$\frac{dx}{dt} = k_1 \left(a - \frac{a+1}{2} x \right) - k'_2 x^2$$

has been solved for given values of k'_2 and k_1 , and it has been shown (in the appendix) that the method is applicable to other cases.

3. Van't Hoff's laws for the variation of equilibrium constants and coefficients of reaction velocity with temperature have been applied to the values of k_1 and K obtained in these experiments, and a close agreement between observed and calculated values has been found.

4. By means of the equations expressing the laws referred to in paragraphs (2) and (3) it is possible to compute the percentage of CO formed at any temperature and with any time of contact.

5. It has been shown that for the production of a high percentage of CO gas, the producer fuel bed should have a temperature of 1300° C. or over, and that increasing the depth of the hot

portion of the bed will increase the percentage of CO, and consequently the capacity of producer at first rapidly and then more and more slowly.

6. To minimize the production of CO in the boiler furnace the fuel bed should be thin. Increasing the velocity of the gas will tend to decrease rather than increase the percentage of CO formed.

APPENDIX

ON THE COMPUTATION OF THE CONSTANTS OF THE REACTION
EQUATION

BY CHARLES N. HASKINS.

1. REDUCTION AND INTEGRATION OF THE DIFFERENTIAL EQUATION.

The differential equation is

$$\frac{dx}{dt} = k_1 \left(a - \frac{a+1}{2} x \right) - k_2 x^2,$$

$$\text{where } 100 a = \% (\text{CO} + \text{CO}_2) \text{ at time } t = 0, \quad (1)$$

$$100 x = \% \text{ CO at time } t,$$

$$t = \text{time in seconds,}$$

and k_1 and k_2 are the two constants of the reaction the values of which are sought. The initial condition is that

$$x = 0, \text{ when } t = 0. \quad (2)$$

To integrate, we introduce a new variable z and new constants α, γ , defined by the relations

$$\left. \begin{aligned} z &= \frac{\left(\frac{a+1}{2}\right) x}{2a - \left(\frac{a+1}{2}\right) x}, & x &= \frac{2 \alpha z}{\left(\frac{a+1}{2}\right)(1+z)}, \\ \alpha &= \frac{1}{2} \left(\frac{a+1}{2}\right) k_1 \sqrt{1 + \left(\frac{2}{a+1}\right)^2 \frac{4a k_2}{k_1}}, & k_1 &= \frac{2 \alpha \gamma}{\left(\frac{a+1}{2}\right)}, \\ \gamma &= \frac{1}{\sqrt{1 + \left(\frac{2}{a+1}\right)^2 \frac{4a k_2}{k_1}}}, & k_2 &= \frac{1}{\alpha} \left(\frac{a+1}{2}\right) \frac{\alpha (1 - \gamma^2)}{2\gamma} \end{aligned} \right\} \quad (3)$$

The differential equation becomes, under these substitutions,

$$\frac{dz}{dt} = \frac{a}{r}(r^2 - z^2) \quad (4)$$

with the initial condition $z = 0$, when $t = 0$. (5)

Integrating, we have

$$\ln^1 \frac{r+z}{r-z} = 2 at; \quad (6)$$

and solving for z ,

$$z = r \frac{e^{at} - e^{-at}}{e^{at} + e^{-at}} = r \tanh at; \quad (7)$$

whence, substituting in (3),

$$x = \frac{2 ar \tanh at}{\left(\frac{a+1}{2}\right)(1+r \tanh at)}. \quad (8)$$

2. THE EQUATION FOR γ AND THE CRITERION FOR THE EXISTENCE OF ONE AND ONLY ONE ROOT.

We have (equation 8) an expression by means of which the per cent of CO at any time t may be computed if the constants γ and a are known. We now wish to determine γ and a from two pairs of observed corresponding values of t and x . Let these two pairs be (t_1, x) , and (t_2, x_2) , and let $t_2 > t_1$.

Then since a is known we may compute

$$z_1 = \frac{\left(\frac{a+1}{2}\right) x_1}{2a - \left(\frac{a+1}{2}\right) x_1}, \quad z_2 = \frac{\left(\frac{a+1}{2}\right) x_2}{2a - \left(\frac{a+1}{2}\right) x_2};$$

and have

$$\ln \frac{r+z_1}{r-z_1} = 2 at_1, \quad \ln \frac{r+z_2}{r-z_2} = 2 at_2, \quad (9)$$

from which γ and a are to be determined. Eliminating a , we readily obtain

$$\ln \frac{r+z_2}{r-z_2} = \frac{t_2}{t_1} \ln \frac{r+z_1}{r-z_1}. \quad (10)$$

The symbols $\ln x$, $\log x$ will be used to denote the natural and the common logarithm of x , respectively.

The determination of γ and α depends therefore on the solution of this (transcendental) equation in γ .

Consideration of the function

$$U(\gamma) \equiv t_1 \ln \frac{\gamma + z_2}{\gamma - z_2} - t_2 \ln \frac{\gamma + z_1}{\gamma - z_1}, \quad (11)$$

and of its derivative

$$U'(\gamma) \equiv -2 \left(\frac{t_1 z_2}{\gamma^2 - z_2^2} - \frac{t_2 z_1}{\gamma^2 - z_1^2} \right) \quad (12)$$

shows that the equation

$$\ln \frac{\gamma + z_2}{\gamma - z_2} = \frac{t_2}{t_1} \ln \frac{\gamma + z_1}{\gamma - z_1} \quad (10)$$

has a root $\gamma > z_2$ when and only when

$$t_2 z_1 - t_1 z_2 > 0, \text{ that is, } \frac{z_2}{z_1} < \frac{t_2}{t_1}. \quad (13)$$

If a root exists there is but one, and it satisfies the inequalities

$$z_2 < \gamma < \sqrt{z_1 z_2 - \frac{(t_2 z_2 - t_1 z_1)}{t_2 z_1 - t_1 z_2}}. \quad (14)$$

The inequality (13) furnishes a negative criterion for the applicability of the differential equation (1) to a reaction under investigation. For if the reaction is governed by equation (1) and if the observations are made with sufficient accuracy there must exist a γ satisfying equation (10) and hence the inequality (13) must be satisfied. If, then, this inequality is not satisfied, and hence no such γ can be found; then either the assumptions involved in (1) must be invalid, or there must be errors in the observations. On the other hand, if (13) is satisfied we can only conclude that (1) *may* be applicable, and we proceed to determine whether it is so by computing γ and α and comparing the values of x computed by means of (8) with the observed values of x .

3. SOLUTION OF THE EQUATION $\ln \frac{\gamma + z_2}{\gamma - z_2} = \frac{t_2}{t_1} \ln \frac{\gamma + z_1}{\gamma - z_1}$.

If the selected pairs (t_1, x_1) , (t_2, x_2) of observed values satisfy the criterion

$$\frac{z_2}{z_1} < \frac{t_2}{t_1} \quad (13)$$

we compute γ as follows. Passing, for convenience of computation, from natural to common logarithms, we have

$$\log \frac{\gamma + z_2}{\gamma - z_2} = \frac{t_2}{t_1} \log \frac{\gamma + z_1}{\gamma - z_1}. \quad (10a)$$

Assume now a value for γ , say $\gamma = \gamma_1$,

where
$$z_2 < \gamma_1 < \sqrt{z_1 z_2 \frac{(t_2 z_2 - t_1 z_1)}{t_2 z_1 - t_1 z_2}}.$$

Then we may compute a quantity $\log N_1$ by the equation

$$\log N_1 = \frac{t_2}{t_1} \log \frac{\gamma + z_1}{\gamma - z_1}, \quad (15)$$

Determine now a new value of γ , $\gamma = \gamma_2$, by the relation

$$\log \frac{\gamma_2 + z_2}{\gamma_2 - z_2} = \log N_1 \quad (16)$$

that is,

$$\frac{\gamma_2 + z_2}{\gamma_2 - z_2} = N_1 \quad (16a)$$

or¹

$$\gamma_2 = z_2 \frac{N_1 + 1}{N_1 - 1}. \quad (16b)$$

Proceed now to determine a new approximation γ_3 from γ_2 in the same way that γ_2 was determined from γ_1 , and continue the process until its repetition produces either no change, or a change which is negligible compared with the experimental errors. It will be found that in general the process converges fairly rapidly and only a few repetitions are necessary.

¹This computation may be abridged by the use of Gaussian logarithms.

Suppose, then, that γ has been found by this process. Then α is computed by either of the relations

$$\alpha = \frac{1}{t_2} \ln \frac{\gamma + z_1}{\gamma - z_1}, \quad \alpha = \frac{1}{t_1} \ln \frac{\gamma + z_2}{\gamma - z_2} \quad (9a)$$

or, what amounts to the same thing, if N is the last of the numbers $N_1, N_2, \dots$ used in computing γ ,

$$\begin{aligned} \alpha &= \frac{1}{t_2} \ln \frac{\gamma + z_2}{\gamma - z_2} = \frac{1}{t_2} \ln N = \frac{1}{t_2} \log N \cdot \ln 10, \\ \alpha &= \frac{2.3026}{t_2} \log N. \end{aligned} \quad (17)$$

4. COMPUTATION OF THE REACTION-CONSTANTS k_1 , k_2 , AND VERIFICATION OF THE REACTION-EQUATION.

When the constants α , γ have been computed we can find the original constants k_1 , k_2 by the relations

$$k_1 = \frac{2 \alpha \gamma}{\left(\frac{a+1}{2}\right)}, \quad k_2 = \frac{1}{a} \left(\frac{a+1}{2}\right)^{\alpha} \frac{(1-\gamma^2)}{\gamma}. \quad (3)$$

To determine the applicability of the reaction-equation (1) to the case in hand, we have now only to introduce the values α , γ , just found into the equation

$$z = \gamma \tanh \alpha t, \quad (7)$$

$$x = \frac{2 \alpha \gamma \tanh \alpha t}{\left(\frac{a+1}{2}\right) \left(1 + \gamma \tanh \alpha t\right)} \quad (8)$$

and compare the values of z or x obtained with those found by observation. For this purpose equation (7) is the simpler, especially when the z 's corresponding to observed values of x have already been computed.

5. CORRECTION OF THE CONSTANTS α, γ BY THE METHOD OF LEAST SQUARES.

The constants α, γ obtained above are determined by two pairs of observations only. It is of course desirable that all the observations be used in fixing their values, as on account of experimental errors the values obtained from different pairs of observations will in general not be identical. We proceed, therefore, to correct the constants by the Method of Least Squares.

Let $(t_1, x_1); (t_2, x_2); \dots (t_n, x_n)$ be n observed pairs of values of t and x . The problem is then to determine α and γ in such a way that if

$$\ln \left(\frac{\gamma + z_i}{\gamma - z_i} \right) - 2 \alpha t_i \equiv \delta_i \quad (6')$$

then

$$n\delta^2 \equiv \sum_1^n \delta_i^2 \quad (18)$$

shall be a minimum.

In order that α and γ shall make $n\delta^2$ a minimum they must satisfy the so-called normal equations:

$$\begin{aligned} \frac{n}{2} \frac{d(\delta^2)}{d\alpha} &\equiv \sum_1^n \delta_i \frac{d\delta_i}{d\alpha} \equiv \sum_1^n 2 t_i \left\{ 2 \alpha t_i - \ln \left(\frac{\gamma + z_i}{\gamma - z_i} \right) \right\} = 0, \\ \frac{n}{2} \frac{d(\delta^2)}{d\gamma} &\equiv \sum_1^n \delta_i \frac{d\delta_i}{d\gamma} \equiv \\ &\sum_1^n \left\{ \frac{1}{\gamma + z_i} - \frac{1}{\gamma - z_i} \right\} \left\{ \ln \left(\frac{\gamma + z_i}{\gamma - z_i} \right) - 2 \alpha t_i \right\} = 0. \end{aligned} \quad (19)$$

These equations may be written

$$\begin{aligned} 2 \alpha \sum_1^n t_i^2 + \sum_1^n t_i \ln \left(\frac{\gamma - z_i}{\gamma + z_i} \right) &= 0, \\ 2 \alpha \sum_1^n \frac{t_i z_i}{\gamma^2 - z_i^2} + \sum_1^n \frac{z_i}{\gamma^2 - z_i^2} \ln \left(\frac{\gamma - z_i}{\gamma + z_i} \right) &= 0. \end{aligned} \quad (20)$$

As their exact solution in their present form is impracticable on account of their complexity we replace them in the usual way by a system of approximately equivalent linear equations, by making use of the fact that the quantities u, v by which a and γ differ from a_0, γ_0 respectively are small compared with a_0, γ_0 .

Substituting

$$\begin{aligned} a &= a_0 + u \\ \gamma &= \gamma_0 + v, \end{aligned} \quad (21)$$

we have

$$\begin{aligned} 2u \sum_1^n t_i^2 + \sum_1^n t_i \ln \left(\frac{1 + \frac{v}{\gamma_0 - z_i}}{1 + \frac{v}{\gamma_0 + z_i}} \right) &= \sum_1^n t_i \left\{ \ln \left(\frac{\gamma_0 + z_i}{\gamma_0 - z_i} \right) - 2a_0 t_i \right\} \\ &\quad + z_i \ln \left(\frac{1 + \frac{v}{\gamma_0 - z_i}}{1 + \frac{v}{\gamma_0 + z_i}} \right) \quad (22) \\ 2u \sum_1^n \frac{t_i z_i}{(\gamma_0^2 - z_i^2) \left(1 + \frac{2\gamma_0 v + v^2}{\gamma_0^2 - z_i^2} \right)} + \sum_1^n \frac{z_i \ln \left(\frac{\gamma_0 + z_i}{\gamma_0 - z_i} \right) - 2a_0 t_i}{(\gamma_0^2 - z_i^2) \left(1 + \frac{2\gamma_0 v + v^2}{\gamma_0^2 - z_i^2} \right)} \\ &= \sum_1^n \frac{z_i \left\{ \ln \left(\frac{\gamma_0 + z_i}{\gamma_0 - z_i} \right) - 2a_0 t_i \right\}}{(\gamma_0^2 - z_i^2) \left(1 + \frac{2\gamma_0 v + v^2}{\gamma_0^2 - z_i^2} \right)}. \end{aligned}$$

Expanding the logarithms by Maclaurin's series and neglecting terms of the order of uv and v^2 in comparison with those of the order of u and v we have

$$\begin{aligned} u \sum_1^n t_i^2 + v \sum_1^n \frac{t_i z_i}{\gamma_0^2 - z_i^2} &= \sum_1^n \frac{t_i}{2} \left(\ln \frac{\gamma_0 + z_i}{\gamma_0 - z_i} - 2a_0 t_i \right) \\ u \sum_1^n \frac{t_i z_i}{\gamma_0^2 - z_i^2} + v \sum_1^n \frac{z_i^2}{(\gamma_0^2 - z_i^2)} &= \sum_1^n \frac{z_i}{(\gamma_0^2 - z_i^2)} \left(\ln \frac{\gamma_0 + z_i}{\gamma_0 - z_i} - 2a_0 t_i \right) \end{aligned} \quad (23)$$

Expressing the natural logarithms in terms of common logarithms we have, putting

$$A_i \equiv t_i, \quad B_i \equiv \frac{z_i}{\gamma_0^2 - z_i^2}, \quad C_i \equiv \log \frac{\gamma_0 + z_i}{\gamma_0 - z_i} - M t_i,$$

$$K \equiv \frac{\ln 10}{2} \equiv 1.15129, \quad M \equiv \frac{2a_0}{\ln 10} \equiv 0.86859 a_0;$$

$$u \sum_1^n A_i^2 + v \sum_1^n A_i B_i = K \sum_1^n A_i C_i, \quad (24)$$

$$u \sum_1^n B_i A_i + v \sum_1^n B_i^2 = K \sum_1^n B_i C_i, \quad (25)$$

or in the usual notation of the method of least squares

$$\begin{aligned} u [A A] + v [A B] &= K [A C], \\ u [B A] + v [B B] &= K [B C]. \end{aligned} \quad (26)$$

From these equations u and v are readily computed and hence the corrected values

$$\begin{aligned} \gamma &= a_0 + u \\ \gamma &= a_0 + v \end{aligned} \quad (27)$$

are found.

6. APPLICATION OF THE METHOD OF §§ 3, 4 TO OTHER EQUATIONS OF REACTION-VELOCITY.

The well known equation¹

$$\frac{dx}{dt} = k_1 (1 - x)^2 - k_2 x^2, \quad (27)$$

with the initial condition $x = 0$, when $t = 0$, is reducible by the substitutions

$$\left. \begin{aligned} z &= \frac{x}{1-x}, & x &= \frac{z}{1+z}, \\ a &= \sqrt{k_1 k_2}, & k_1 &= a \gamma, \\ \gamma &= \sqrt{\frac{k_1}{k_2}}, & k_2 &= \frac{a}{\gamma}, \end{aligned} \right\} \quad (28)$$

to the form we have considered, viz.:

¹Cf Nernst, *Theoretische Chemie*, 5 Aufl, p. 564; 2d English edition, p. 568.

$$\frac{dz}{dt} = \frac{a}{\gamma} (\gamma^2 - z^2) \quad (4)$$

with the initial condition $z = 0$ when $t = 0$. Its integral is, therefore,

$$\ln \frac{\gamma + z}{\gamma - z} = 2 at, \quad (6)$$

or

$$z = \gamma \tanh at, \quad (7)$$

or

$$x = \frac{\gamma \tanh at}{1 + \gamma \tanh at}, \quad (8)$$

and the constants are determined by the equation

$$\ln \frac{\gamma + z_2}{\gamma - z_2} = \frac{t_2}{t_1} \ln \frac{\gamma + z_1}{\gamma - z_1} \quad (10)$$

if the criterion

$$\frac{z_2}{z_1} < \frac{t_2}{t_1} \quad (13)$$

is satisfied.

The more general equation¹

$$\frac{dx}{dt} = k_1 (a_1 - x) (b_1 - x) - k_2 (a_2 + x) (b_2 + x), \quad (29)$$

with initial conditions $x = 0$, when $t = 0$ is reducible by a substitution of the form

$$x = \frac{\rho z - \sigma}{1 + z}. \quad (30)$$

where ρ and σ are constants depending on a_1, b_1, a_2, b_2 but not on k_1, k_2 , to the equation

$$\frac{dz}{dt} = \frac{a}{\gamma} (\gamma^2 - z^2). \quad (4)$$

The initial conditions, however, are now not

$$t = 0, z = 0, \quad (5)$$

but

$$t = 0, z = \frac{\sigma}{\rho} \equiv z_0; \quad (5')$$

and hence the integral and the equation determining the constants are more complicated.

¹Cf. Nernst, *Theoretische Chemie*, 5 Aufl. p. 543; 2d English edition, p. 542.

The equation determining γ is

$$\ln \frac{\gamma + z_2}{\gamma - z_2} = \frac{t_2}{t_1} \ln \frac{\gamma + z_1}{\gamma - z_1} - \left(\frac{t_2}{t_1} - 1 \right) \ln \frac{\gamma + z_0}{\gamma - z_0} \quad (31)$$

and the criterion for the existence of a solution is

$$t_2 z_1 - t_1 z_2 - (t_2 - t_1) z_0 > 0. \quad (32)$$

The solution, if existent, is unique, and is determined by a process similar to that of § 3.

7. CONCLUSION.

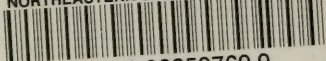
The analysis in the preceding sections furnishes a simple negative criterion (13) or (32) for the applicability of the differential equation (1) or (29) to a given reaction and, in case this criterion is satisfied, provides a straightforward method of computing the numerical values of the reaction constants k_1 and k_2 from *any* two pairs of observed corresponding values of x and t . It renders unnecessary, moreover, except as a matter of control, any observations of equilibrium conditions.

The detailed discussion of the more general equation (29) is reserved for subsequent publication.

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- Bulletin No. 25.* Lighting Country Homes by Private Electric Plants, by T. H. Amrine. 1908.
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